MASTER

DRAINAGE CRITERIA MANUAL

TOWN OF ADDISON

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DRAINAGE CRITERIA MANUAL
FOR
THE TOWN OF ADDISON, TEXAS

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PREFACE

The purpose of this Drainage Criteria Manual is to provide a written summary of the Town of Addison's general drainage policies, along with the assumptions upon which those policies are based and the procedures by which those policies are to be implemented. The Manual is designed to minimize developer/engineer processing and review time by clarifying the Town's design criteria, submittal requirements and procedures. The developer/engineer is encouraged to enter discussions with Town of Addison staff early in the development process.

This Drainage Criteria Manual will be updated periodically. Developers and engineers are encouraged to monitor proposed changes and keep his Manual up to date.

1.0 GENERAL DRAINAGE POLICY

1.1 GENERAL

The acceptance of the design, size, type and location of all storm drainage facilities in the Town of Addison is subject to the approval of the Town Engineer.

The criteria and specifications presented in this manual are to be considered as minimum requirements.

All storm drainage facilities in the Town of Addison shall be designed for the case of ultimate watershed development based on a 100-year frequency design storm.

In the case of drainage improvements required for private development projects, the developer and his engineer shall bear total responsibility for the adequacy of the design. The approval of a given drainage facility by the Town of Addison in no way relieves the developer and his engineer from their responsibility.

Prior to any channel improvement or stormwater detention design, the Town Engineer shall be consulted regarding preferred flood control strategies for the watershed of interest.

1.2 REQUIRED TECHNICAL INFORMATION TO BE SUBMITTED FOR CITY REVIEW

The design engineer shall submit appropriate hydraulic and hydrologic design calculations and other technical information for review by the Town Engineer, to demonstrate that the minimum standards provided in this manual have been met.

The design engineer may be required to submit additional technical information beyond that which is required in this manual if it is considered necessary by the Town Engineer to complete the design review.

1.3 CRITERIA FOR FILLING IN A FLOODPLAIN

The criteria established for filling in a floodplain as described in this section shall apply to any construction within a Special Flood Hazard Area.

A Special Flood Hazard Area is defined as the land within the floodplain which is subject to a 1% or greater chance of flooding in any given year. This area has been identified in part by the Federal Emergency Management Agency (FEMA) on its Flood Insurance Rate Map (FIRM), Community No. 481089, dated July 16, 1980. On watercourses not covered by the FIRM, the Special Flood Hazard Area shall be determined by an appropriate floodplain analysis. The results of such an analysis must be reviewed and accepted by the Town Engineer.

The following specifications shall be observed:

1. There shall be no increase in the water surface elevation on any property upstream, downstream or on the opposite bank from the proposed site caused by construction activity in the floodplain. The property owner/developer shall be required to provide technically acceptable proof (such as a backwater analysis) that this restriction has not been violated.

2. Any increase in mean streamflow velocity shall be limited so as not to exceed the open channel velocity limitations delineated in Section 7.0 of this manual. In addition, there shall be no increase in erosional activity on any property upstream, downstream or on the opposite bank from the proposed site caused by construction activity in the floodplain. The owner/developer shall be required to provide technically acceptable proof (such as a backwater analysis) that this restriction has not been violated.

3. The toe of any fill slope must parallel the natural channel to prevent an unbalanced streamflow in the improved channel.

4. To ensure maximum accessibility to the improved channel area for maintenance and other purposes, and to lessen the probability of slope erosion during periods of high water, maximum slopes of the filled area shall not exceed three (horizontal) to one (vertical) without approval of the Town Engineer. The slope of any excavated area in the channel which is not in rock may not exceed four to one without approval of the Town Engineer. Vertical walls, terracing and other slope treatments may be considered provided no unbalancing of streamflow results, and only as a part of a landscaping plan submission.

5. A landscaping plan may be required by the City Manager which includes details for erosion control of cut and filled slopes, restoration of excavated areas and tree protection where possible in and below the filled area.

6. Fill must be placed to at least 1 foot above the 100-year flood line based on fully-developed discharges. The lowest floor of any structure must be constructed at least 2 feet above the 100-year flood line based on fully-developed discharges.

1.4 HYDRAULIC CRITERIA FOR BRIDGE DESIGN

Wherever possible, the proposed bridge should be designed to span a channel section equal to the approaching channel section. Wherever possible, bridges should be constructed to cross channels at a 90° angle, which normally will result in the most economical construction. Wherever the bridge structure is skewed, the bents should be constructed parallel to the flow of water.

Velocities through the bridge opening shall not exceed 6 feet per second (fps) for earthen channels unless certified geotechnical data is provided which confirms that the channel banks will withstand higher velocities. Appropriate slope protection shall be provided for the channel slopes and the bottom when velocities are higher than 6 fps or the erosive velocity as determined by geotechnical data.

A distance of 1 foot between the maximum 100-year design water surface, based on a fully-developed discharge, and the lowest point of the bridge stringers should be maintained.

For a detailed discussion of the hydraulics of bridge design, the engineer is referred to the U.S. Department of Transportation/Federal Highway Administration publication, Hydraulics of Bridge Waterways, Revised March, 1978.

2.0 DETERMINATION OF STORM RUNOFF

2.1 GENERAL

Long-term records of rainfall and the resulting runoff in an area provide the best data source on which to base the design of storm drainage and flood control systems for that area. However, it is not possible to obtain these records in sufficient quantities in most watersheds. Therefore, numerous procedures have been developed to attempt to effectively relate a given amount of rainfall in a given physiographic area to a given pattern of runoff. This section provides the acceptable procedures for the determination of flood hydrographs in the Town of Addison.

For drainage areas larger than 400 acres, the determination of flood hydrographs in the Town of Addison shall be accomplished through the use of one of two computer programs, HEC-1 or NUDALLAS. Both programs employ acceptable hydrologic methodologies and are discussed in this section. The use of alternative means of flood hydrograph determination is acceptable subject to the approval of the Town Engineer and provided full documentation of the alternative methodology is presented for review.

For drainage areas less than 400 acres, it is acceptable to utilize the Rational Method in cases where determination of the flood peak alone is required. A detailed discussion of the Rational Method is presented in this Section 2.5.

2.2 EFFECT OF URBANIZATION

It is generally accepted that urban development has a pronounced effect on the rate and volume of runoff from a given rainfall. Urbanization generally alters the hydrology of a watershed by improving its hydraulic efficiency, reducing its surface infiltration and reducing its storage capacity. The reduction of a watershed's storage capacity and surface infiltration results from the replacement of porous surfaces and ponding areas with buildings, streets, parking lots, sidewalks and other facilities characteristic of urban development.

Zoning maps, future land use maps and watershed master plans should be used as aids in establishing the anticipated surface character following development. The selection of design runoff coefficients and/or percent impervious cover factors must be based upon the appropriate degree of future urbanization.

2.3 RAINFALL-RUNOFF DESIGN FREQUENCY

All drainage structures or improvements in the Town of Addison shall be designed to properly accommodate the runoff from a storm event of 100-year frequency.

2.4 RELATING RAINFALL TO RUNOFF

The process of relating rainfall on a watershed to runoff at a given point in the watershed is generally accomplished in the following four discrete stages:

1. Determination of the Design Rainfall;
2. Calculation of Abstractions (Losses);
3. Generation of the Runoff Hydrograph for the Subarea; and
4. Determination of the Change in the Shape of the Hydrograph (termed routing) as the Flood Wave Moves Through the Watershed.

Each of these four stages will be discussed in the following sections (Sections 2.4.1 through 2.4.4).

2.4.1 Design Storm Rainfall

1. Parameters of a Storm Event--A design storm rainfall event is described in terms of four parameters: frequency, total storm duration, distribution of intensity with time, and areal extent.

   a. Frequency--Storm frequency is the measure of the expected recurrence interval of a storm of a given magnitude. As an example, the 100-year frequency storm event (with a 100-year frequency magnitude) can be expected to occur on average once every 100 years. In other words, the 100-year storm event has a 1% chance of occurring in any given year.

   b. Total Storm Duration--Total storm duration is defined as the time elapsed between the start and the finish of a rainfall associated with a storm event. The engineer's choice for design storm duration is generally dependent on the size of the pertinent watershed. For design purposes, a storm of 24-hour duration can be expected to adequately reflect most time-related effects on the runoff hydrograph. The storm duration will vary for designation of the probable maximum flood (PMF) precipitation as required by special designs such as dam breach analyses.

   c. Distribution of Intensity with Time--For design purposes in the Town of Addison, the synthetic rainfall hyetograph peak shall occur at the two-thirds point of the total storm duration. Both HEC-1 and NUDALLAS have routines for generating acceptable design rainfall hyetographs.

   d. Areal Extent--The intensity/duration/frequency relationships used to build rainfall hyetographs are based on rainfall amounts measured at a single location. Logically, the larger the watershed being studied, the less rainfall volume per unit area can be expected to fall uniformly over the watershed for a given frequency event.

Figure 15 in the National Weather Service's (NWS) Technical Paper No. 40 presents a means of reducing rainfall totals for a given frequency event as drainage area size increases.

In addition, both the NUDALLAS and HEC-1 programs have the capability to modify runoff hydrographs to account for progressively smaller design storm volumes as areal coverage increases.

2. Intensity/Duration/Frequency Curves--NWS Technical Paper No. 40 (TP-40), "Rainfall Frequency Atlas of the United States," May 1961, provides accumulated point rainfall amounts for storm durations from 30 minutes to 24 hours and event frequencies from 1 to 100 years. National Oceanic and Atmospheric Administration (NOAA) publication NOAA Hydro-35, "Five- to 60-Minute Precipitation Frequency for the Eastern and Central United States," June 1977, provides accumulated point rainfall amounts for durations of 5 minutes, 15 minutes and 1 hour, for 1- and 100-year frequency events. NWS Technical Paper No. 49 (TP-49), "Two- to Ten-Day Precipitation for Return Periods of 2 to 100 Years in the Contiguous United States," 1964, extends the duration range to 10 days.

   TP-40 and Hydro-35 were used to generate Table 2-1, which presents point rainfall amounts for varying durations and frequencies in the Addison, Texas area. Table 2-1 may be used for determining the rainfall volumes for various durations and frequencies to be input to HEC-1 or NUDALLAS. With this data, each program will generate an acceptable design rainfall hyetograph.

TABLE 2-1

POINT RAINFALL AMOUNT FOR VARYING DURATIONS AND FREQUENCIES IN
ADDISON, TEXAS

Duration	1-Yr	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	500-Yr	SPF
5-minute		0.50	.584	.648	.742	.816	.890		
10-minute		.824	.971	1.08	1.24	1.37	1.49		
15-minute		1.05	1.24	1.38	1.59	1.75	1.91		
30-minute	1.20	1.46	1.78	2.02	2.38	2.65	2.92	4.19	
1-hour	1.58	1.88	2.51	2.90	3.39	3.84	4.30	5.28	
2-hour	1.81	2.28	3.01	3.55	4.22	4.71	5.22	6.43	
3-hour	2.00	2.50	3.30	3.90	4.60	5.20	5.77	7.26	
6-hour	2.41	2.96	3.96	4.70	5.58	6.20	6.95	8.69	
12-hour	2.84	3.48	4.70	5.57	6.45	7.39	8.38	10.71	
24-hour	3.24	4.02	5.44	6.40	7.58	8.60	9.63	12.32	
48-hour		4.60	6.08	7.20	8.68	9.70	11.03		
72-hour									15.00
96-hour		5.36	7.10	8.27	9.90	11.23	12.56		

Sources: National Weather Service (NWS). 1961. NWS Technical Paper No. 40, "Rainfall Frequency Atlas of the United States."

National Weather Service (NWS). 1964. NWS Technical Paper No. 49, "Two- to Ten-Day Precipitation for Return Periods of 2 to 100 Years in the Contiguous United States."

National Oceanic and Atmospheric Administration. 1977. NOAA Hydro-35, "Five- to 60-Minute Precipitation Frequency for the Eastern and Central United States."

Note: This table to be used with either the HEC-1 or NUDALLAS methods.

2.4.2 Design Storm Losses

Only a portion of the rainfall volume which falls on a watershed during a storm event actually ends up as stream runoff. The remainder is intercepted by infiltration, depression storage, evaporation and other mechanisms. The volume of rainfall which becomes runoff is termed the "excess" rainfall. The difference between the observed total rainfall hyetograph and the excess rainfall hyetograph is termed abstractions or losses.

Numerous methodologies are available to calculate abstractions. For the Town of Addison, the following loss routines can be used:

- Uniform Loss Rate;
- HEC Exponential Loss Rate;
- Soil Conservation Service (SCS) Curve Number Loss Rate; and
- Holtan Loss Rate.

The default condition for losses as calculated in NUDALLAS makes use of the Block and uniform loss rate methodology. The specific default values contained in the program for the initial loss and average hourly losses for varying frequency events are contained in Table 2-2.

The use of these rates is acceptable in the Town of Addison. The above loss rates in Table 2-2 should also be used as a guideline for application of the uniform loss methodology in HEC-1.

For discussion of all of the above methodologies, the engineer is referred to "HEC-1, Flood Hydrograph Package, User's Manual" and "NUDALLAS, Documentation and Supporting Appendixes."

TABLE 2-2

NUDALLAS BLOCK AND UNIFORM LOSS RATE
RECOMMENDED VALUES FOR ADDISON, TEXAS

	Sand		Clay	
Event Frequency	Initial Loss (in.)	Avg. Hourly Loss (in.)	Initial Loss (in.)	Avg. Hourly Loss (in.)
1-Year	2.1	0.26	1.5	0.2
2-Year	2.1	0.26	1.5	0.2
5-Year	1.8	0.21	1.3	0.16
10-Year	1.5	0.18	1.12	0.14
25-Year	1.3	0.15	0.95	0.12
50-Year	1.1	0.13	0.84	0.1
100-Year	0.9	0.10	0.75	0.07
500-Year	0.6	0.08	0.5	0.05
SPF	0.6	0.08	0.5	0.05

Source: NUDALLAS, Documentation and Supporting Appendixes, U.S. Army Corps of Engineers, Fort Worth District.

2.4.3 Design Storm Runoff

Given the design storm excess rainfall, it is necessary to determine the storm runoff hydrograph for the subarea of interest. In the Town of Addison, flood hydrographs will be determined utilizing the Snyder's unit hydrograph methodology.

1. Snyder's Unit Hydrograph--In a study of watersheds of widely varying size, Snyder found consistent relationships for unit hydrographs in watersheds in which the time to peak (t_p) is 5.5 times longer than the duration of the unit hydrograph excess rainfall (t_r). For a "standard" unit hydrograph in which t_p=5.5t_r, the following relationships were found to hold:

$$t_p = C_t \, (L \, L_{ca})^{0.3}$$

$$q_p = 640 \, \frac{C_p}{t_p}$$

where:

t_r	=	duration of the unit hydrograph excess rainfall (hrs)
t_p	=	time from center of unit hydrograph excess rainfall duration to peak of unit hydrograph (hrs)
L	=	length of main stream (miles)
L_{ca}	=	distance from the point of interest/subarea outlet to the point on the stream nearest the centroid of the watershed area (miles)
C_t	=	timing coefficient representing variation of subarea slope and storage
q_p	=	peak flow rate per unit of drainage area (cfs)
C_p	=	peaking coefficient dependent upon units and drainage basin characteristics such as flood wave and storage

When the "standard" relationship t_p=5.5t_r does not hold, the time to peak (t_p) and the peak discharge per unit area (q_p) can be adjusted for the desired unit hydrograph computation interval t_r (required) as follows:

$$t_p \text{ (adjusted)} = t_p + 0.25 \left(t_r \text{ [required]} - \frac{t_p}{5.5}\right)$$

$$q_p \text{ (adjusted)} = \frac{640\,(C_p)}{t_p}$$

2. Determination of Snyder's Unit Hydrograph Shape--The Snyder's methodology is incapable of yielding a continuous and complete description of the unit hydrograph shape. This task is handled differently by the HEC-1 and NUDALLAS programs.

   In HEC-1, determination of the continuous shape of the unit hydrograph is carried out with an iterative procedure using the Clark unit hydrograph methodology. In NUDALLAS, determination of the shape is based on a method described in James A. Constant's paper, "A Mathematical Determination of the Ordinates of the Unit Hydrograph," 1970.

3. Application of Snyder's Unit Hydrograph in HEC-1 and NUDALLAS--HEC-1 and NUDALLAS handle application of the Snyder's unit hydrograph slightly differently:

   NUDALLAS was originally written to allow the engineer to apply basic hydrologic principles specifically in the Dallas-Fort Worth area with a minimum of input. Consequently, empirical percent urbanization curves, specific to the Dallas area, were developed and internalized in the program. These are used in the determination of the Snyder's parameters, t_p, and are presented in Figures 2-1 and 2-2 at the conclusion of this section.
   In NUDALLAS, the engineer is required to specify the following watershed parameters:

   a. L, the length of the main stream in miles.

b. L_{ca}, the distance in miles from the point of interest/subarea outlet to the point on the stream nearest the centroid of the watershed area in miles.

c. The drainage area size in square miles.

d. S_{st}, the weighted slope of the main drainage course in feet per mile.

e. The percent urbanization is defined as the percent of the subarea which has been developed and improved with channelization or a storm collection network. The engineer is referred to Table 2-3 for a summary of land use types vs. percent urbanization.

f. The percent imperviousness is defined as the percent of the subarea that is covered with impervious material and is hydraulically connected to the subarea's drainage network. The engineer is referred to Table 2-3 for a summary of land use types vs. percent impervious cover.

g. The percent of sand (and, consequently, the percent of clay) is to be determined from local soil surveys.

HEC-1 requires two Snyder's parameters--t_p and C_p. These will be determined as follows:

a. C_p--It has been determined that in the Addison area, C_p shall have a value of 0.719. The parameter C_p is sometimes labeled as C_p 640, and its value for the Addison area is 460.0

b. t_p--To maintain continuity with the NUDALLAS program, t_p will be determined using the percent urbanization curves developed by the U.S. Army Corps of Engineers (Corps) and presented at the conclusion of this section of the manual. This is carried out as follows:

TABLE 2-3

PERCENT URBANIZATION AND IMPERVIOUSNESS SUMMARY
WITH ASSOCIATED LAND USE CATEGORIES
RECOMMENDED VALUES

	Title	Percent Imperviousness	Percent Urbanization
(R-1)	Single Family (12,000 sq. ft. lot)	38	70
(R-2)	Single Family (9,000 sq. ft. lot)	56	80
(R-3)	Single Family (7,500 sq. ft. lot)	70	85
(R-16)	Single Family (16,000 sq. ft. lot)	30	60
	Multi-Family Residential	80	95
	Retail	95	95
	Strip Commercial	90	95
	Shopping Centers	95	95
	Institutional - School, Church, Government	40	70
	Industrial	90	95
	Street Pavement	100	100
	Parks and Developed Open Space	6	10
	Cropland	3	5
	Grassland	0	0
	Woodlands, Forest	0	0
	Water Bodies	100	100
	Barren Land, Gravel Pits	0	0

Sources: Determination of Percent Urbanization/Imperviousness in Watersheds, May 1, 1986, U.S. Army Corps of Engineers.

Urban Hydrology for Small Watersheds, Soil Conservation Service Technical Release No. 55.

- Step 1--Determine what percent of the watershed has clay soils and what percent has sand soils.
- Step 2--Determine the value of

$$\frac{L\ L_{ca}}{(S_{st})^{1/2}}$$

where L, L_{ca} and S_{st} are as described above.

- Step 3--Using the curve for clay (Figure 2-1), determine the lag time (hours) for the given percent urbanization. Using the curve for sand (Figure 2-2), determine the lag time (hours) for the given percent urbanization. Multiply each value of the lag by its respective percent of occurrence and sum them to determine the weighted lag t_p.

2.4.4 Routing the Flood Hydrograph

As a flood wave passes downstream through a channel or detention facility, its shape is altered due to the effects of storage. The procedure for determining how the shape of the flood hydrograph changes is termed flood routing.

1. Stream Routing vs. Reservoir Routing--Flood routing can be classified into two broad but related categories: open channel stream routing and reservoir routing. Reservoir routing is often used to determine the effect of stormwater detention on a runoff hydrograph. Open channel stream routing is used to determine the changing shape of the runoff hydrograph as a function of the channel geometry or storage capacity.

2. Available Methodologies--HEC-1 and NUDALLAS contain several acceptable flood routing routines. For work in the Town of Addison, it is recommended that the engineer employ either the Muskingum method or the Modified Puls method.

a. Muskingum Method--The Muskingum method models the storage volume as a combination of wedge storage caused by a non-level water surface along the routing reach and prism storage formed by a volume of constant cross-section along the length of the prismatic channel. Unlike the Modified Puls method, it is not limited by the assumption of a level pool in the routing reach. The Muskingum parameters K and X are required in HEC-1 and shall be determined as defined in this section of the manual.

The Muskingum model solves the following linear version of the continuity equation:

$$S = K [XI + (I-X) 0]$$

where:

	S	=	storage in the routing reach;
	I	=	inflow rate;
	O	=	outflow rate; and
	K	=	a proportionality parameter which equals the travel time through the routing reach of an incremental flood wave. K may be estimated for use in HEC-1 as follows:

- in wide rectangular channels, K in seconds = (length of the reach in feet) divided by (1.67 x [average velocity of flow in feet per second]). Note that in HEC-1, K must be converted to hours.
- in wide parabolic channels, K in seconds = (length of the reach in feet) divided by (1.44 x [average velocity of flow in feet per second]). Note that in HEC-1, K must be converted to hours.
- in triangular channels, K in seconds = (length of the reach in feet) divided by (1.33 x [average velocity of flow in feet per second]). Note that in HEC-1, K must be converted to hours.

(Note: In HEC-1, see "User's Manual" for limitations on size of K.)

X = a storage parameter which varies between 0 and 0.5. When an incremental change in the inflow to the routing reach has virtually no effect on the outflow (as in a reservoir), X should equal 0. When an incremental change in the inflow will produce a virtually identical immediate change in the outflow (as in a pipe flowing full), X should equal 0.5. For most natural channels, X should range between 0.1 and 0.3.

The overall routing reach may require partitioning into subreaches when channel geometry changes significantly or if the limits of K (see HEC-2 "User's Manual") cannot be satisfied.

For a discussion of the Muskingum routing methodology, the engineer is referred to the HEC-1 "User's Manual."

b. Modified Puls Method--The Modified Puls method is based upon a solution of the simple continuity equation:

$$I - O = \text{change in } S$$

where:

I = inflow;

O = outflow; and

S = storage.

The engineer must provide as input an estimate of starting conditions and storage vs. discharge values as calculated either by a backwater model such as HEC-2, Water Surface Profiles, or, as in the case of a reservoir, by the characteristics of the outlet works.

The overall routing reach should be partitioned into smaller subreaches when significant changes in channel geometry warrant it.

For a detailed discussion of the Modified Puls methodology, the engineer is referred to the "Handbook of Applied Hydrology," Ven Te Chow, 1964 or "NUDALLAS, Documentation and Supporting Appendixes."

2.5 RATIONAL METHOD

2.5.1 General

The Rational Method represents an accepted method for determining peak storm runoff rates for small watersheds that have a drainage system unaffected by complex hydrologic situations such as ponding areas, storage basins and watershed transfers (overflows) of storm runoff. It is generally recommended that the Rational Method be used for areas less than 400 acres in the Town of Addison.

2.5.2 Definition of Rational Formula

The Rational Method is based on the direct relationship between rainfall and runoff, and is expressed by the following equation:

$$Q = CiA$$

where:

- Q is defined as the peak rate of runoff in cubic feet per second (cfs). Actually, Q is in units of inches per hour per acre. Since this rate of in/hr/ac differs from cfs by less than 1%, the more common cfs is used.

- C is the runoff coefficient, a dimensionless coefficient of runoff which represents the ratio of peak discharge to rainfall intensity (i).

- i is the average intensity of rainfall in inches per hour for a period of time equal to the critical time of flow concentration for the drainage area to the point of design.

A is the area in acres contributing runoff to the point of design during the critical time of concentration.

2.5.3 Assumptions of Rational Method

Basic assumptions associated with the Rational Method are:

1. The computed peak rate of runoff at the design point is a function of the average rainfall rate during the time of concentration to that point.

2. The frequency or recurrence interval of the peak discharge is equal to the frequency of the average (uniform) rainfall intensity associated with the critical time of concentration (duration).

3. The ratio of runoff to rainfall, C, is uniform during the storm duration.

4. Rainfall intensity is uniform during the storm duration.

5. The contributing area is the area that drains to the design point within the critical time of concentration.

2.5.4 Runoff Coefficient (C)

In relating peak rainfall rates to peak discharges, the runoff coefficient "C" in the Rational Formula is dependent on the character of the drainage area's surface. The rate and volume of a storm's rainfall that reaches an area's storm sewer system depends on the relative porosity (imperviousness), ponding character, slope and conveyance properties of the surface. Soil types, vegetation conditions and impervious surfaces, such as pavements and buildings, are among the major determining factors in selecting an area's "C" factor. Onsite inspections and aerial photographs may prove valuable in estimating the nature of the surface within the drainage area.

It should be noted that the runoff coefficient "C" is the variable of the Rational Method which is least susceptible to precise determination. Proper use requires judgment and experience on the part of the engineer.

Zoning and land use maps should be used to select the appropriate type of development activity which exists, or is proposed, in a drainage area. Coefficients for specific land use types can be used to develop a composite runoff coefficient based in part on the percentage of different types of land uses in the drainage area.

Table 2-4 presents recommended ranges for "C" values for various land uses and specific surface types for the 100-year frequency storm. Runoff coefficients should always be selected based on a fully-urbanized drainage area.

2.5.5 Rainfall Intensity

Rainfall intensity (i) is the average rainfall rate in inches per hour which is considered for a particular area or subarea, and is selected on the basis of design rainfall duration and design frequency of occurrence. The design rainfall duration is equal to the critical time of concentration for all portions of the drainage area under consideration that contribute flow to the design point during the critical time of concentration. All drainage structures in the Town of Addison will be designed to accommodate the 100-year design storm. The design rainfall intensity to be used in the Rational equation is determined for a given duration and frequency from the frequency/duration/intensity curves presented in Figure 2-3.

Runoff from a watershed usually reaches a peak at the time when the entire drainage area is contributing, in which case the time of concentration is the time for water to flow from the most remote point in the watershed to the design point.

The time of concentration to any point in a storm drainage system is a combination of the "inlet time" and the "time of flow in the conduit."

TABLE 2-4

RUNOFF COEFFICIENTS FOR RATIONAL FORMULA

Type of Drainage Area	Runoff Coefficient
Business:	
Downtown Areas	0.70-0.95
Neighborhood Areas	0.65-0.90
Residential:	
Single-family Areas	0.35-0.70
Multi-units, detached	0.50-0.75
Multi-units, attached	0.65-0.80
Apartment Dwelling Areas	0.70-0.85
Industrial:	
Light Areas	0.60-0.80
Heavy Areas	0.70-0.90
Parks, Cemeteries	0.20-0.50
Playgrounds	0.30-0.60
Unimproved Areas:	
Sand or Sandy Loam Soil, 0-3%	0.15-0.20
Sand or Sandy Loam Soil, 3-5%	0.20-0.25
Black or Loessial Soil, 0-3%	0.18-0.25
Black or Loessial Soil, 3-5%	0.25-0.30
Black or Loessial Soil, >5%	0.30-0.50
Deep Sand Area	0.10-0.15
Steep Grassed Slopes	0.70
Lawns:	
Sandy Soil, flat, 2%	0.05-0.10
Sandy Soil, average, 2-7%	0.10-0.15
Sandy Soil, steep, 7%	0.15-0.20
Heavy Soil, flat, 2%	0.13-0.17
Heavy Soil, average, 2-7%	0.18-0.25
Heavy Soil, Steep, 7%	0.25-0.35

TABLE 2-4 (Concluded)

Type of Drainage Area	Runoff Coefficient
Streets:	
Asphaltic	0.75-0.95
Concrete	0.85-0.95
Brick	0.70-0.90
Drives and Walks	0.75-0.90
Roofs	0.80-0.95

The inlet time is the time for water to flow over the surface to the storm sewer inlet. Inlet time decreases as the slope and the imperviousness of the surface increase, and it increases as the distance over which the water has to travel increases and as retention by the contact surfaces increases. Average velocities for estimating travel time for overland flow can be calculated using Figure 2-4.

The inlet time shall be determined by direct computation using the following formula:

$$T = \frac{D_f}{60V}$$

where:

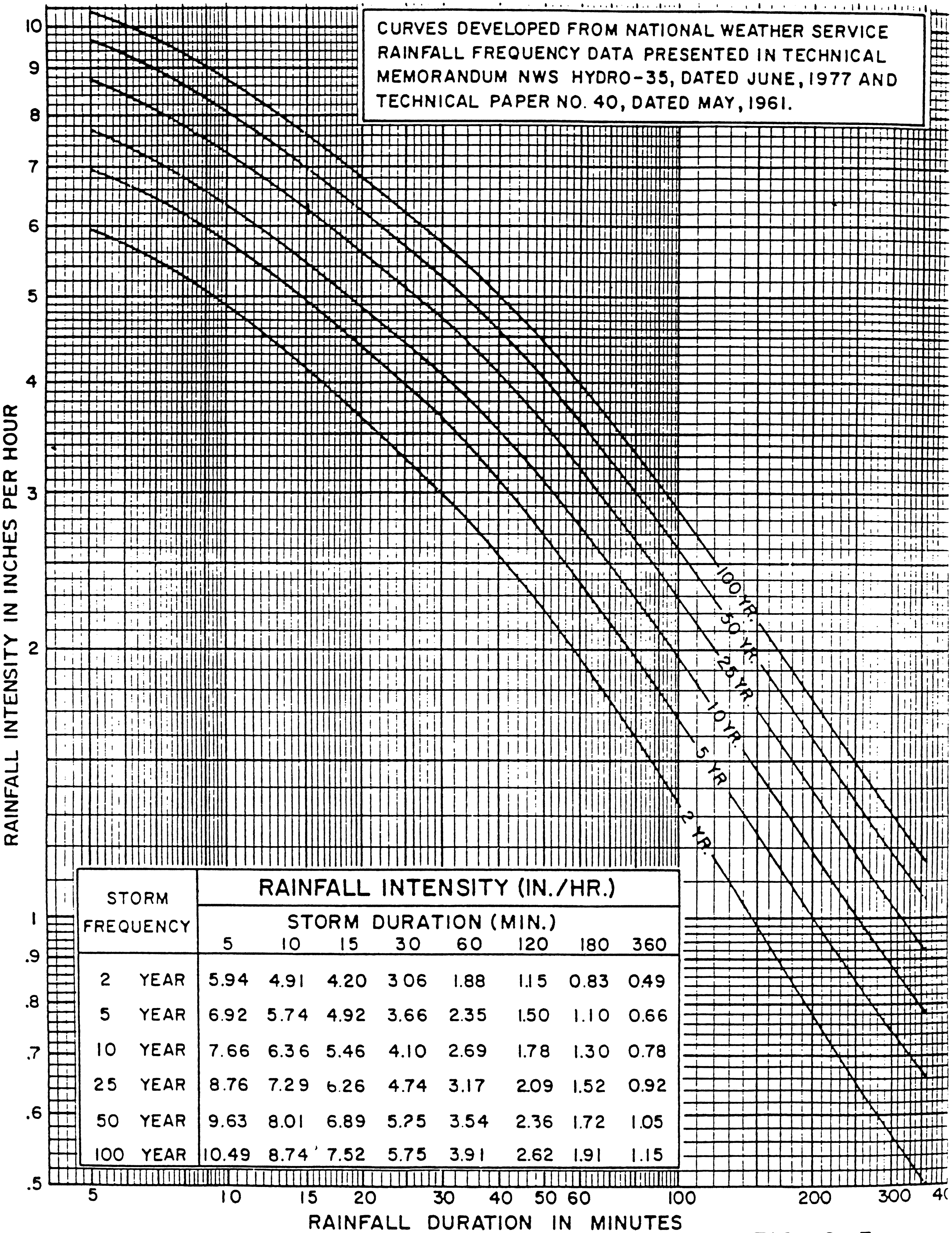
	T	=	inlet or overland flow time (minutes);
	D_f	=	flow distance (feet); and
	V	=	average velocity of runoff flow (ft/sec).

The designer should consider the future development of an area when selecting an appropriate inlet time for that area.

The time of flow in the conduit is the quotient of the length of the conduit and the velocity of flow as computed using the hydraulic characteristics of the conduit.

2.5.6 Drainage Area (A)

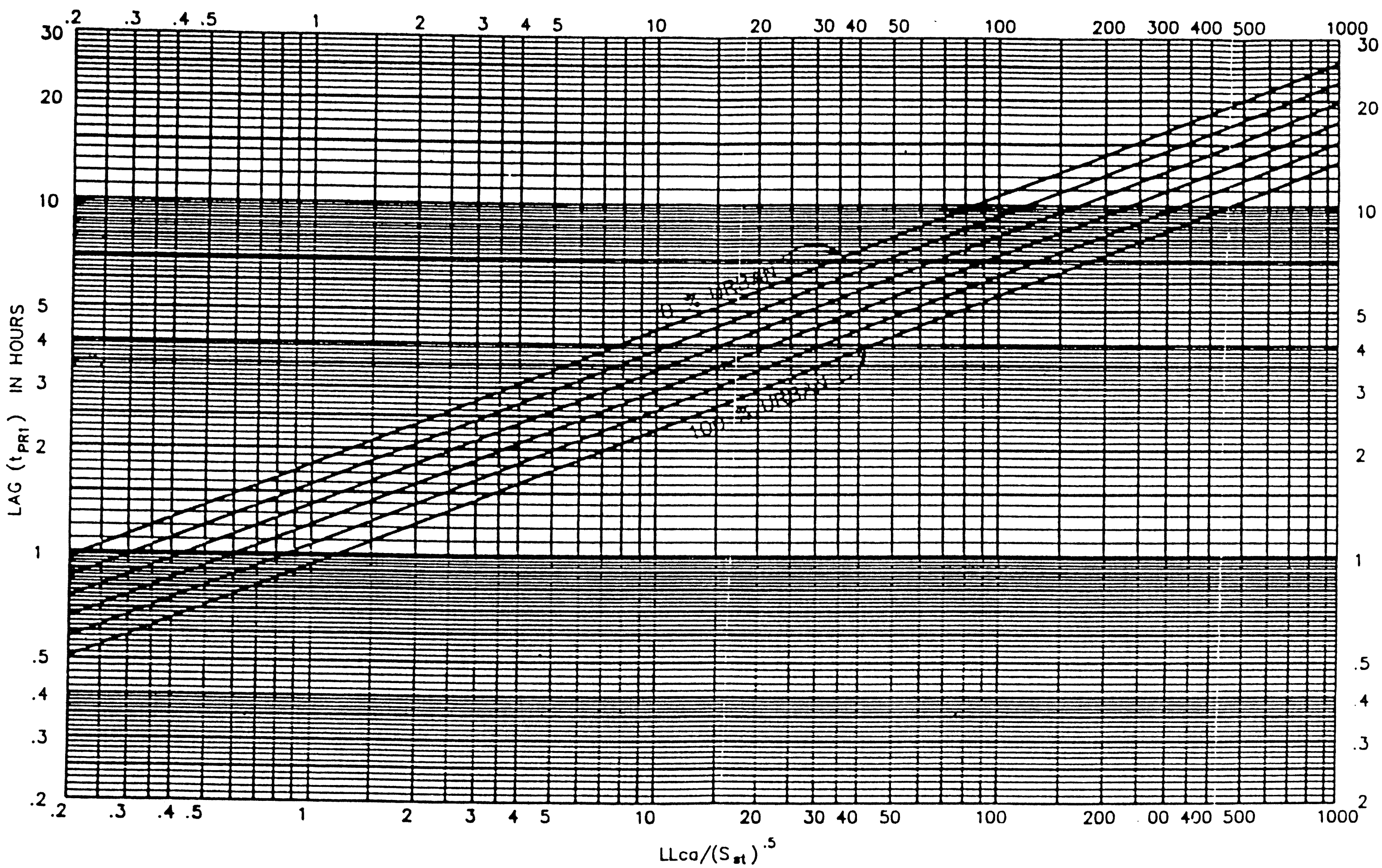
The size and shape of the drainage area may be determined through the use of topographic maps, supplemented by field surveys where topographic data has changed or where the contour interval is too great to distinguish the direction of flow. A drainage area map will be provided for each project. The drainage area contributing to the system being designed and the drainage subarea contributing to each inlet point shall be identified. The outlines of the drainage divides must follow actual lines rather than the artificial land divisions. The drainage divide lines are determined by the pavement slopes, locations of downspouts, grading of lawns and other features that are introduced by the urbanization process.



STORM FREQUENCY	RAINFALL INTENSITY (IN./HR.) STORM DURATION (MIN.) 5	10	15	30	60	120	180	360
2 YEAR	5.94	4.91	4.20	3.06	1.88	1.15	0.83	0.49
5 YEAR	6.92	5.74	4.92	3.66	2.35	1.50	1.10	0.66
10 YEAR	7.66	6.36	5.46	4.10	2.69	1.78	1.30	0.78
25 YEAR	8.76	7.29	6.26	4.74	3.17	2.09	1.52	0.92
50 YEAR	9.63	8.01	6.89	5.25	3.54	2.36	1.72	1.05
100 YEAR	10.49	8.74	7.52	5.75	3.91	2.62	1.91	1.15

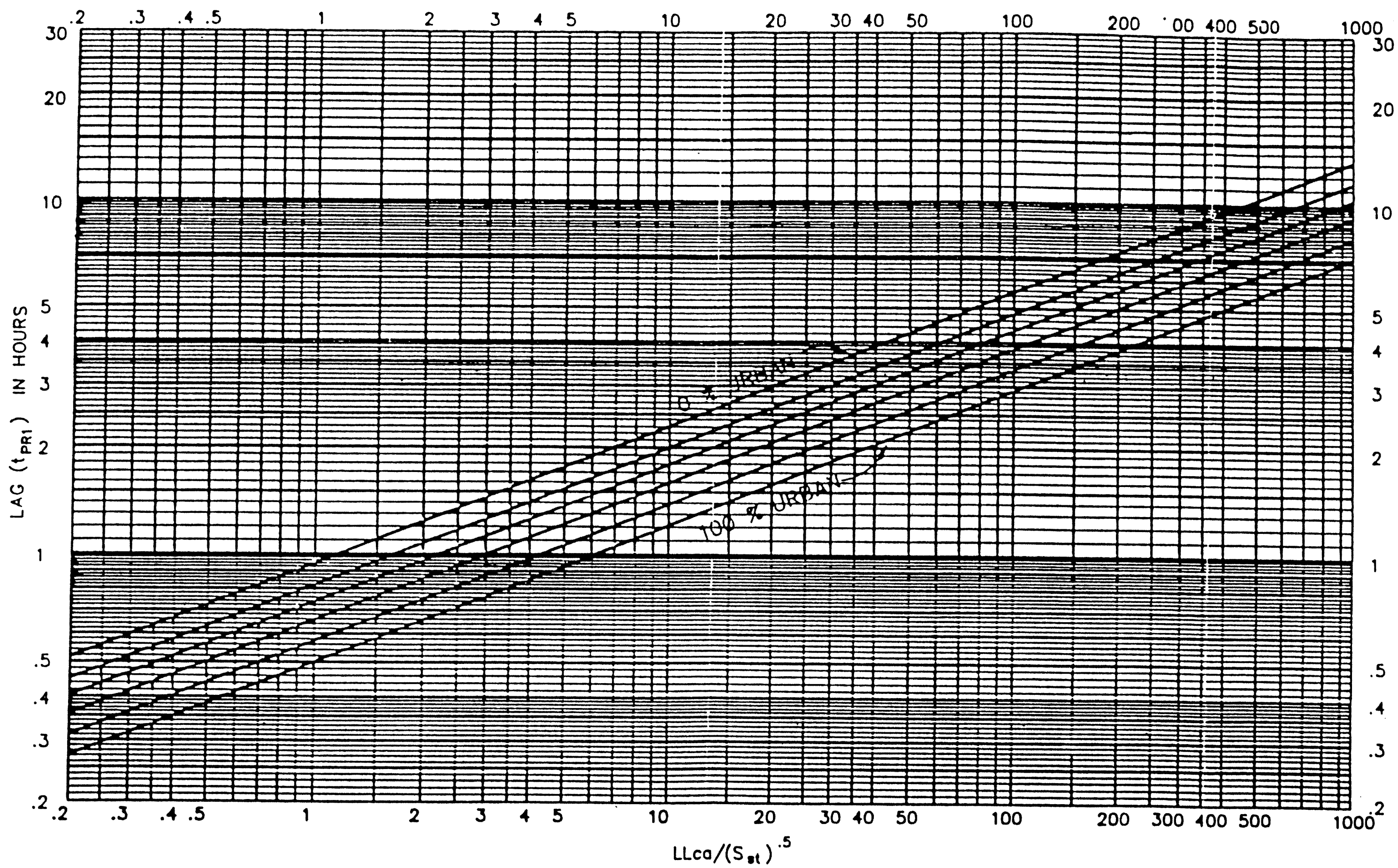
FIG. 2-3

RAINFALL CURVES FOR ADDISON, TEXAS



ESPEY, HUSTON & ASSOCIATES, INC.
Engineering & Environmental Consultants

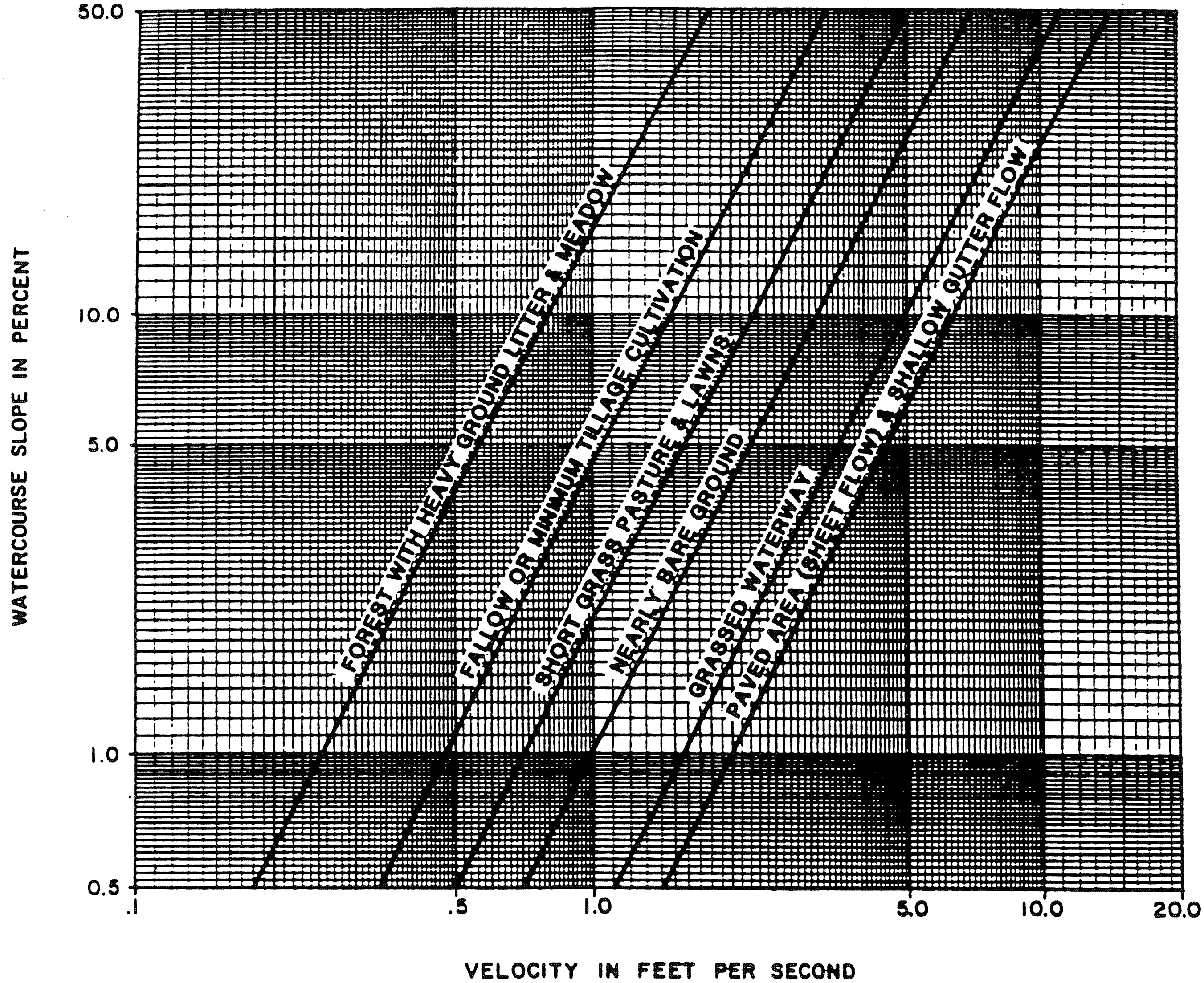
NUDALLAS
SAND URBANIZATION
CURVE
FIG. 2-2
SOURCE. U.S. ARMY ENGINEER DISTRICT, FT. WORTH



ESPEY, HUSTON & ASSOCIATES, INC.
Engineering & Environmental Consultants

NUDALLAS
CLAY URBANIZATION
CURVE
FIG 2-1

SOURCE: U.S. ARMY ENGINEER DISTRICT
FT. WORTH



AVERAGE VELOCITIES FOR ESTIMATING TRAVEL TIME
FOR OVERLAND FLOW

FOR ADDISON, TEXAS

FIG. 2-4

3.0 FLOW IN STREETS

3.1 GENERAL

The permissible flow of water in the streets should be related to the allowable extent and frequency of interference to traffic and the likelihood of flood damage to surrounding property. Interference to traffic is regulated by design limits on the spread of water into traffic lanes.

The design storm to be used for determining the flow of stormwater in streets shall be the 100-year design storm.

Curbs shall be a minimum height of 6" measured from the flow line of the gutter to the top of curb.

3.1.1 Interference Due to Flow in Streets

Water which flows in a street, whether from rainfall directly onto the pavement surface or overland flow entering from adjacent land areas, will flow in the gutters of the street until it reaches an overflow point or some outlet, such as a storm sewer inlet. As the flow progresses downhill and additional areas contribute to the runoff, the width of flow will increase and progressively encroach into the traffic lanes. Field observations show that vehicles will crowd adjacent lanes to avoid curb flow.

As the width of flow increases further, it reaches a point where the street loses its effectiveness as a traffic carrier. During these periods, it is imperative that emergency vehicles such as fire trucks, ambulances and police cars be able to traverse the street.

3:1.2 Interference Due to Ponding

Storm runoff ponded on the street surface because of grade changes or the crown slope of intersecting streets has a substantial effect on the street carrying capacity. Because of the localized nature of ponding, vehicles moving at a relatively high speed may enter a pond. The manner in which ponded water affects traffic is essentially the same as for curb flow; that is, the width of spread into the traffic lanes is critical. In the case of low points, the designer shall ensure that the spread of water in the street does not exceed permissible levels on either side of the low point.

3.1.3 Interference Due to Water Flowing Across Traffic Lanes

Whenever storm runoff, other than limited sheet flow, moves across a traffic lane, a serious and dangerous impediment to traffic flow occurs. The cross-flow may be caused by superelevation of a curve, a street intersection, overflow from the higher gutter on a street with cross-fall, or simply poor street design.

The depth and velocity of cross-flows shall be maintained within such limits that they will not have sufficient force to threaten moving traffic.

3.1.4 Effect on Pedestrians

In areas with heavily used sidewalks, splash due to vehicles moving through water adjacent to the curb is a serious problem.

Streets should be classified with respect to pedestrian traffic as well as vehicular traffic. As an example, streets which are classified as residential for vehicles and located adjacent to a school are arterials for pedestrian traffic. Allowable width of gutter flow and extent of ponding should reflect this fact.

3.2 PERMISSIBLE SPREAD OF WATER

3.2.1 Primary Arterial Streets (With A Median)

Inlets shall be spaced at such an interval as to provide a minimum of one clear traffic lane in each direction during the peak flows of the 100-year design storm, provided that flow in the gutter does not exceed 5 inches.

3.2.2 Minor Arterial Streets (Without A Median)

The flow of water in gutters of a minor arterial street shall be limited so that two standard lanes will remain clear during the peak runoff from the 100-year design storm, provided the depth of flow does not exceed the top of curb.

3.2.3 Collector Streets

The flow of water in gutters of a collector street shall be limited so that one standard lane will remain clear during the peak runoff from the 100-year design storm, provided the depth of flow does not exceed the top of curb.

3.2.4 Residential Streets

The spread of water in the gutters of a residential street shall be limited such that one 12-foot traffic lane remains clear during the peak runoff from the 100-year design storm, provided the depth of flow does not exceed the top of curb. Inlets shall be located at least every 600 feet, or wherever the gutter flow exceeds the permissible spread of water.

3.3 MINIMUM AND MAXIMUM VELOCITIES

To ensure cleaning velocities at very low flows, the street gutter shall have a minimum longitudinal slope of 0.004 feet/foot (0.4%). The maximum velocity of curb flow shall be 10 fps.

Along sharp horizontal curves, peak flows tend to jump behind the curb line at driveways and other curb breaks. Water running behind the curbline can result in considerable damage due to erosion and flooding. In a gutter which has a slope greater than 0.60 feet/foot (6%) and a radius of 400 feet or less, the 100-year design flow shall not exceed 4 inches at the curb.

3.4 DESIGN METHOD

3.4.1 Straight Crowns

Flow in gutters which are on straight crown pavements is normally calculated by using Manning's Equation for uniform flow in pavement gutters or triangular channels. The equation is:

$$Q_o = 0.56 \frac{z}{n} (S_o)^{1/2} (Y_o)^{8/3}$$

where:

where:	Q_o	=	gutter discharge (cfs);
	z	=	reciprocal of the crown slope (ft/ft);
	S_o	=	street or gutter slope (ft/ft);
	n	=	roughness coefficient;
	Y_o	=	depth of flow in gutter (ft).

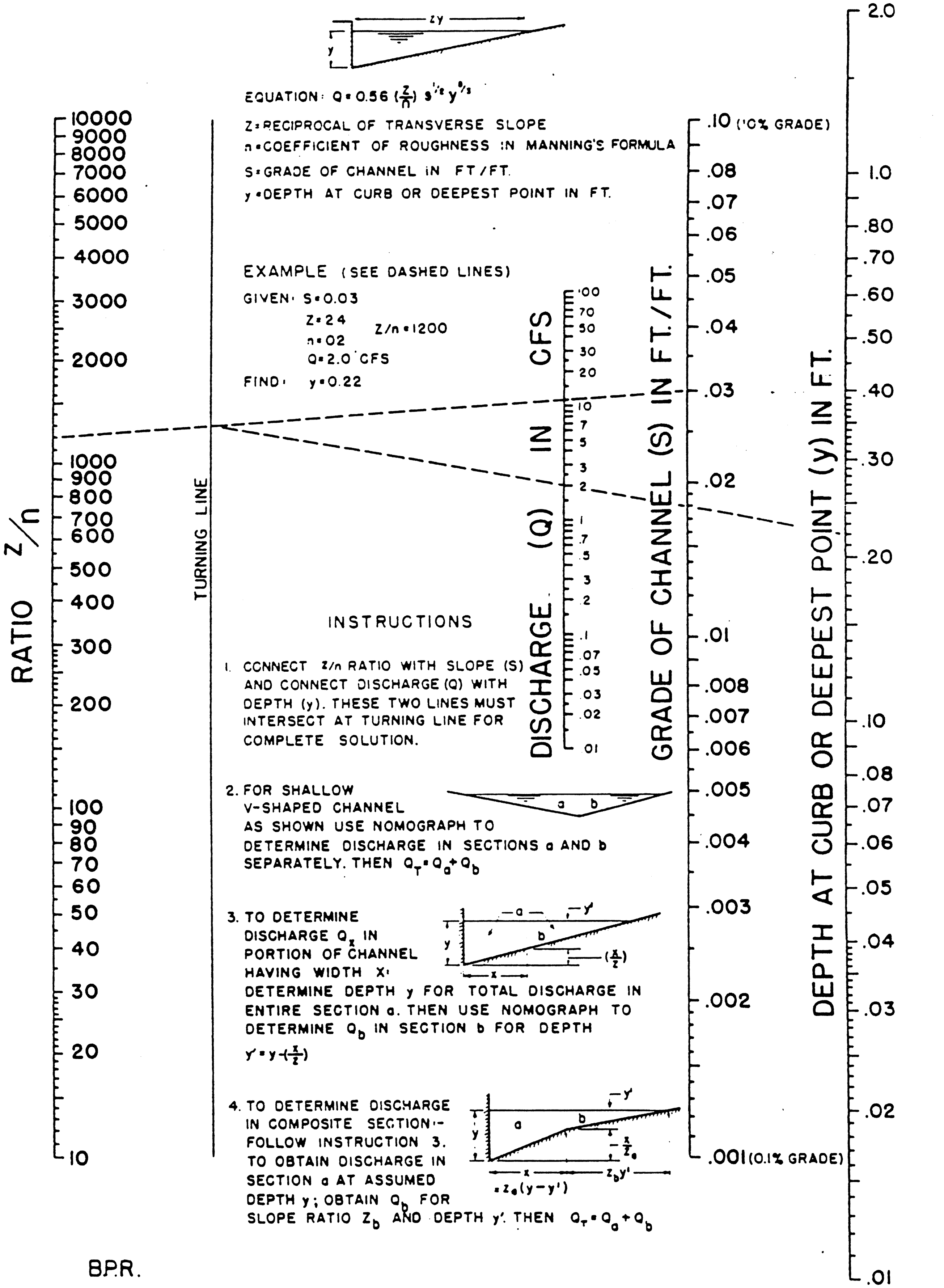
The nomograph in Figure 3-1 provides for direct solution of flow conditions for triangular channels most frequently encountered in urban drainage design. For the usual concrete gutter, a value of 0.017 for "n" is recommended.

3.4.2 Alleys

The flows created by the 100-year design storm shall be contained within the capacity of all paved alleys.

Alley capacities shall be checked at all alley turns and "T" intersections to determine if curbing is needed or grades should be flattened.

Curbing shall be required for a sufficient length on either side of a curb inlet to contain the flow within the alley pavement limits.



NOMOGRAPH FOR FLOW
IN TRIANGULAR CHANNELS

Figure 3-1

4.0 STORM DRAIN INLETS

4.1 GENERAL

The primary purpose of storm drain inlets is to intercept excess surface runoff and deposit it in a drainage system, thereby reducing the possibility of surface flooding.

The most common location for inlets is in streets which collect and channelize surface flow, making it convenient to intercept. Because the primary purpose of streets is to carry vehicular traffic, inlets must be designed so as not to conflict with that purpose.

The following guidelines shall be used in the design of inlets to be located in streets:

1. When recessed inlets are used, they shall not interfere with the intended use of the sidewalk.
2. Design and location of inlets shall take into consideration pedestrian and bicycle traffic.
3. Use of depressed inlets shall not be allowed unless approved by the Town Engineer.
4. All points of concentration from offsite drainage shall be picked up by an inlet or headwall before flow enters the street or crosses the sidewalk.
5. Inlets shall generally be provided to intercept runoff on the upstream side of street intersections. Wherever possible, the end of the inlet shall be located 10 feet from the curb radius.
6. The design storm to be used for all inlets and other stormwater collection facilities shall be the 100-year design storm.
7. Inlets shall be located so as to provide minimum interference with adjacent property.
8. Inlet locations directly above storm sewer lines shall be avoided.

9. Inlet design and location must be compatible with the criteria established in Section 3.0 of this manual.

4.2 CLASSIFICATION

Inlets are classified into two major groups: inlets in low points and inlets on grade. Each group of inlets includes several varieties. The following are presented herein and are likely to find reasonably wide use (see Table 4-1).

- Inlets in Low Points
  1. Curb Opening (recessed and non-recessed)
  2. Grate
  3. Combination (grate and curb opening)
  4. Drop
  5. Drop (grate covering)

- Inlets on Grade
  1. Curb Opening (recessed and non-recessed)
  2. Grate
  3. Combination (grate and curb opening)

Inlet computations must be submitted to the Town Engineer as part of the plans convenient for review and permanent record.

4.3 INLETS IN LOW POINTS

Inlets shall be located in all street low points to relieve ponding, unless the Town Engineer approves an alternative outlet for the stormwater. The capacity of inlets in low points must be known in order to determine the depth and width of ponding for a given discharge.

TABLE 4-1

STORM DRAIN INLETS

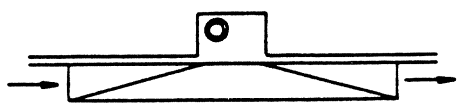
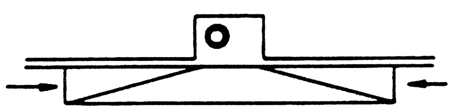
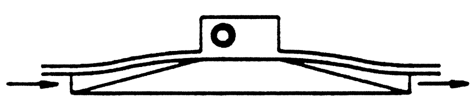
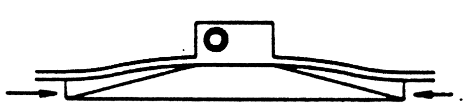
Inlet Description	Avail. Inlet Sizes	
 STANDARD CURB OPENING INLET ON GRADE	4' 5' 6' 8' 10' 14'	Residential Street
 STANDARD CURB OPENING INLET AT LOW POINT	4' 5' 6' 8' 10' 14'	Residential Street
 RECESSED CURB OPENING INLET ON GRADE	4' 5' 6' 8' 10' 14'	Collector Streets Minor Arterials Primary Arterials
 RECESSED CURB OPENING INLET AT LOW POINT	4' 5' 6' 8' 10' 14'	Collector Streets Minor Arterials Primary Arterials

TABLE 4-1 (Cont'd)

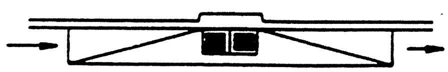
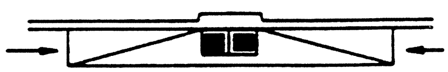
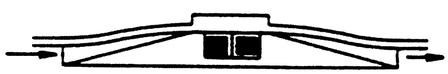
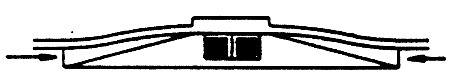
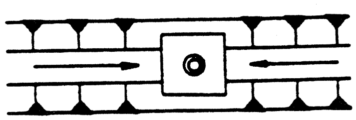
Inlet Description	Avail. Inlet Sizes	
 COMBINATION INLET ON GRADE	5' 10'	Combination Inlets to be Used Where Space Behind Curb Prohibits Other Inlets
 COMBINATION INLET AT LOT POINT	5' 10'	Combination Inlets to be Used Where Space Behind Curb Prohibits Other Inlets
 COMBINATION RECESSED INLET ON GRADE	5' 10'	Collector Streets Minor Arterials Primary Arterials
 COMBINATION RECESSED INLET AT LOW POINT	5' 10'	Collector Streets Minor Arterials Primary Arterials

TABLE 4-1 (Concluded)

Inlet Description	Avail. Inlet Sizes	
 GRATE INLETS	2 Grate 3 Grate 4 Grate 6 Grate	Grate Inlets to be Used Where Space Restrictions Prohibit Other Inlet Types or at Locations With No Curb
 DROP INLET	2' x 2' 3' x 3' 4' x 4'	Open Channels

4.3.1 Curb Opening Inlets and Drop Inlets

Unsubmerged curb opening inlets and drop inlets in low points are considered to function as rectangular weirs with a coefficient of discharge of 3.0. Their capacity shall be based on the following equation:

$$Q = 3.0y^{3/2}L$$

where:
- Q = capacity of curb opening inlet or drop inlet (cfs);
- y = head at the inlet (feet);
- L = length of opening through which water enters the inlet.

Where the depth of water is such that the curb inlet is completely submerged, the proper orifice formula should be used in determining the discharge rather than the weir formula. This condition is rare, as the location of inlets should be such as to preclude ponding in sufficient depth to submerge the inlet.

The capacity of a low point inlet may be determined by use of Figure 4-1, which has been developed from the preceeding equation.

4.3.2 Grate Inlets

The flow of water through grate openings may be treated in the same manner as the flow of water through rectangular orifices with a coefficient of discharge of 0.60. The capacity can then be computed with the following equation:

$$Q = 4.82Ay^{1/2}$$

where: Q = the discharge in cfs;

A = the area of orifice (the net area of the openings in the grate) in square feet;

y = head on grate in feet.

This formula gives the theoretical capacity of the grate inlet. Since grate inlets are subject to considerable clogging, it is recommended that for practical purposes, the capacity of the grate inlet be taken as one-half of the value given by this formula, or conversely that the net area of the grate be twice as large as the theoretical area required when calculated by the above formula.

The capacity of a grate inlet in a low point may be determined by use of Figure 4-2, which has been developed from the preceeding equation.

4.3.3 Combination Inlets

The capacity of a combination inlet consisting of a grate and curb opening inlet in a low point shall be considered to be 50% of the sum of the capacities as determined for a curb opening inlet and a grate inlet which operate independently (allowing for reduction due to clogging). When the capacity of the gutter is not exceeded, the grate inlet accepts the major portion of the flow. Under severe flooding conditions, however, the curb inlet will accept most of the flow since its capacity varies with $y^{3/2}$, whereas the capacity of the grate inlet varies as $y^{1/2}$.

4.4 INLETS ON GRADE

4.4.1 Curb Opening Inlets

The capacity of a curb inlet, like any weir, depends upon the head and length of overfall. In the case of an undepressed curb opening inlet, the head at the upstream end of the opening is the depth of flow in the gutter. Undepressed inlets do not interfere with traffic and usually are not susceptible to clogging.

The capacity of an undepressed inlet shall be determined by the use of Figures 4-3 and 4-4.

4.4.2 Grate Inlets on Grade

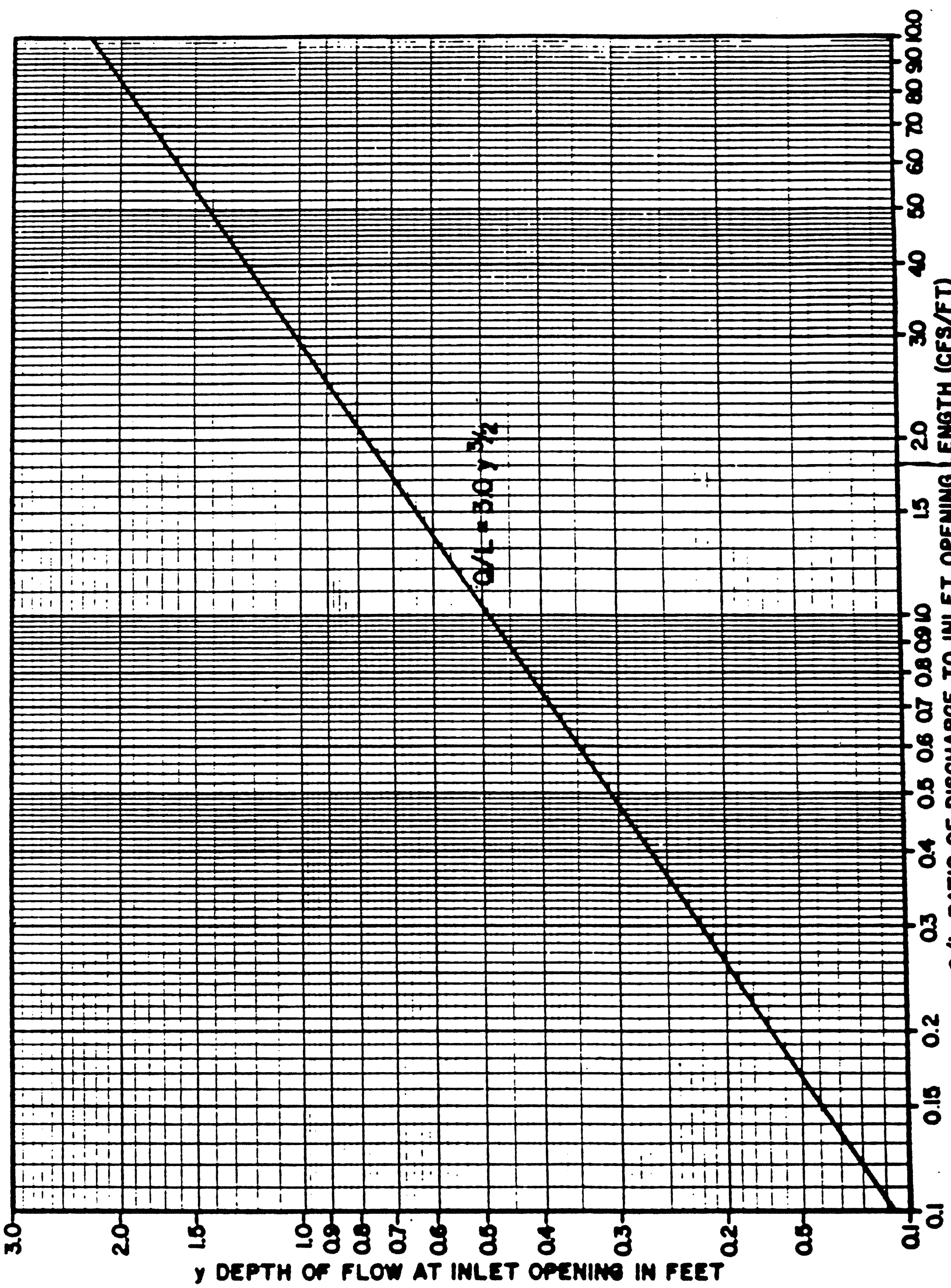
Undepressed grate inlets on grade have a greater hydraulic capacity than curb inlets of the same length so long as they remain unclogged. Undepressed grate inlets on grade are inefficient in comparison to grate inlets in low points.

Grates with bars parallel to the curb should always be used for the above described installations because transverse framing bars create splash, which causes the stormwater to jump or ride over the grate. Grates used shall be certified by the manufacturer as bicycle-safe. The capacity of a grate inlet on grade may be determined by the use of Figure 4-2 with a reduction of 25% due to clogging.

4.4.3 Combination Inlets on Grade

Undepressed combination (curb opening and grate) inlets on grade have greater hydraulic capacity than curb inlets or grate inlets of the same length. Generally speaking, combination inlets are the most efficient of the three types of undepressed inlets presented in this manual. Grates with bars parallel to the curb should always be used. The basic difference between a combination inlet and a grate inlet is that the curb opening receives the carryover flow that falls between the curb and the grate.

The capacity of a combination inlet shall be considered to be 50% of the sum of the capacities as determined for a curb opening inlet and a grate inlet (without allowing for reduction due to clogging).

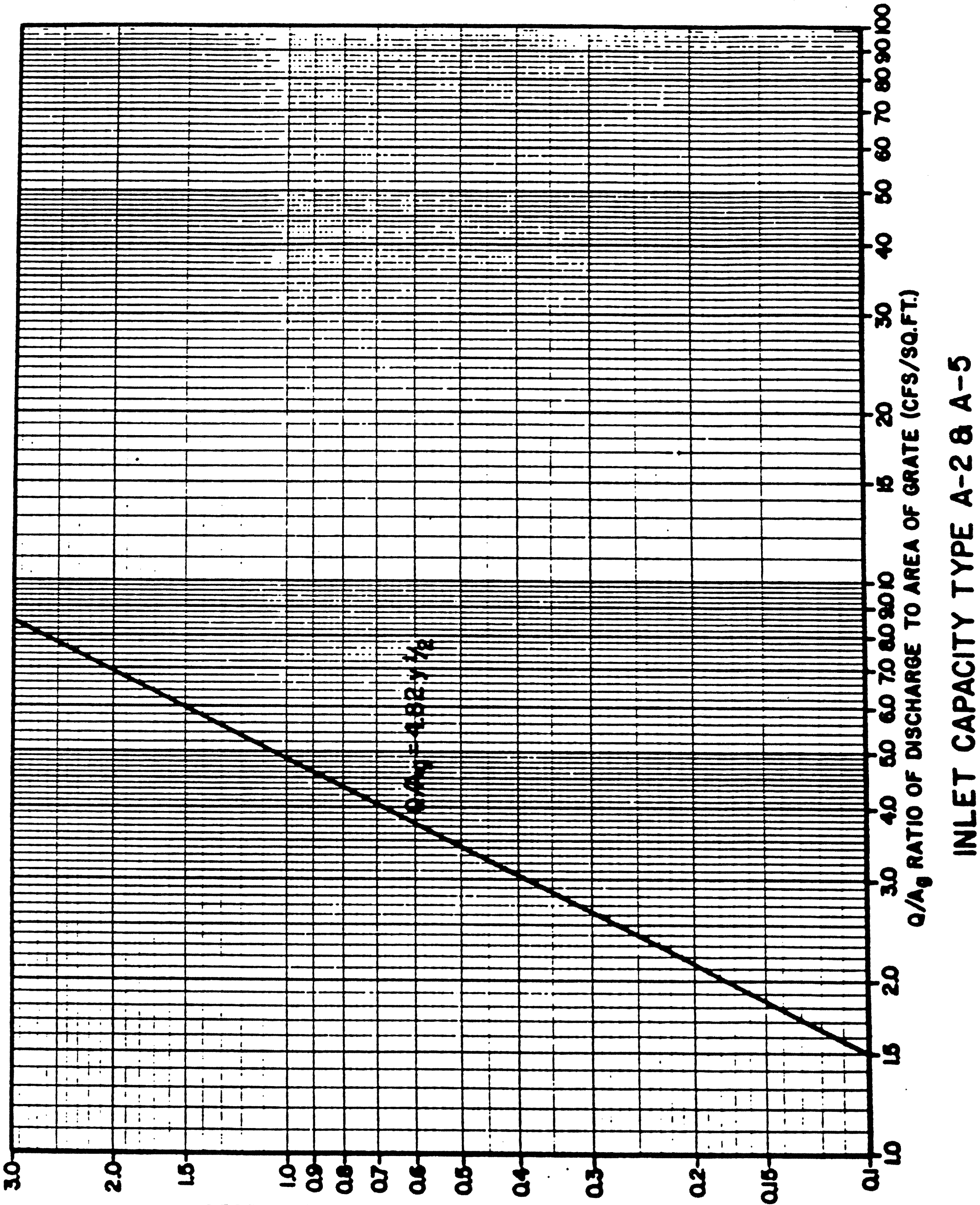


INLET CAPACITY TYPE A-1 & A-4

CAPACITY OF

CURB INLET OPENING IN A LOW POINT

Figure 4-1



CAPACITY OF
GRATE INLET IN A LOW POINT

Figure 4-2

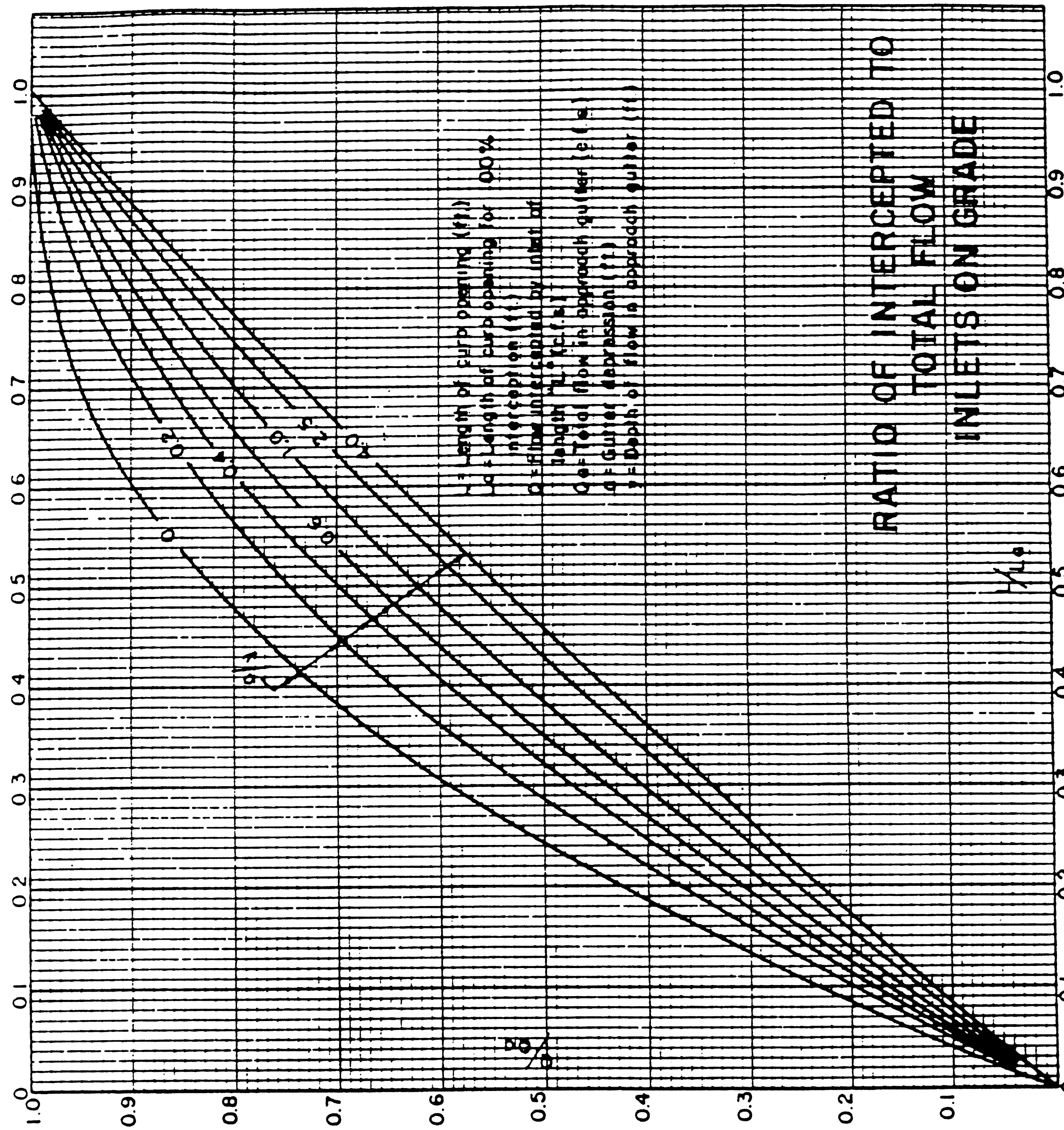
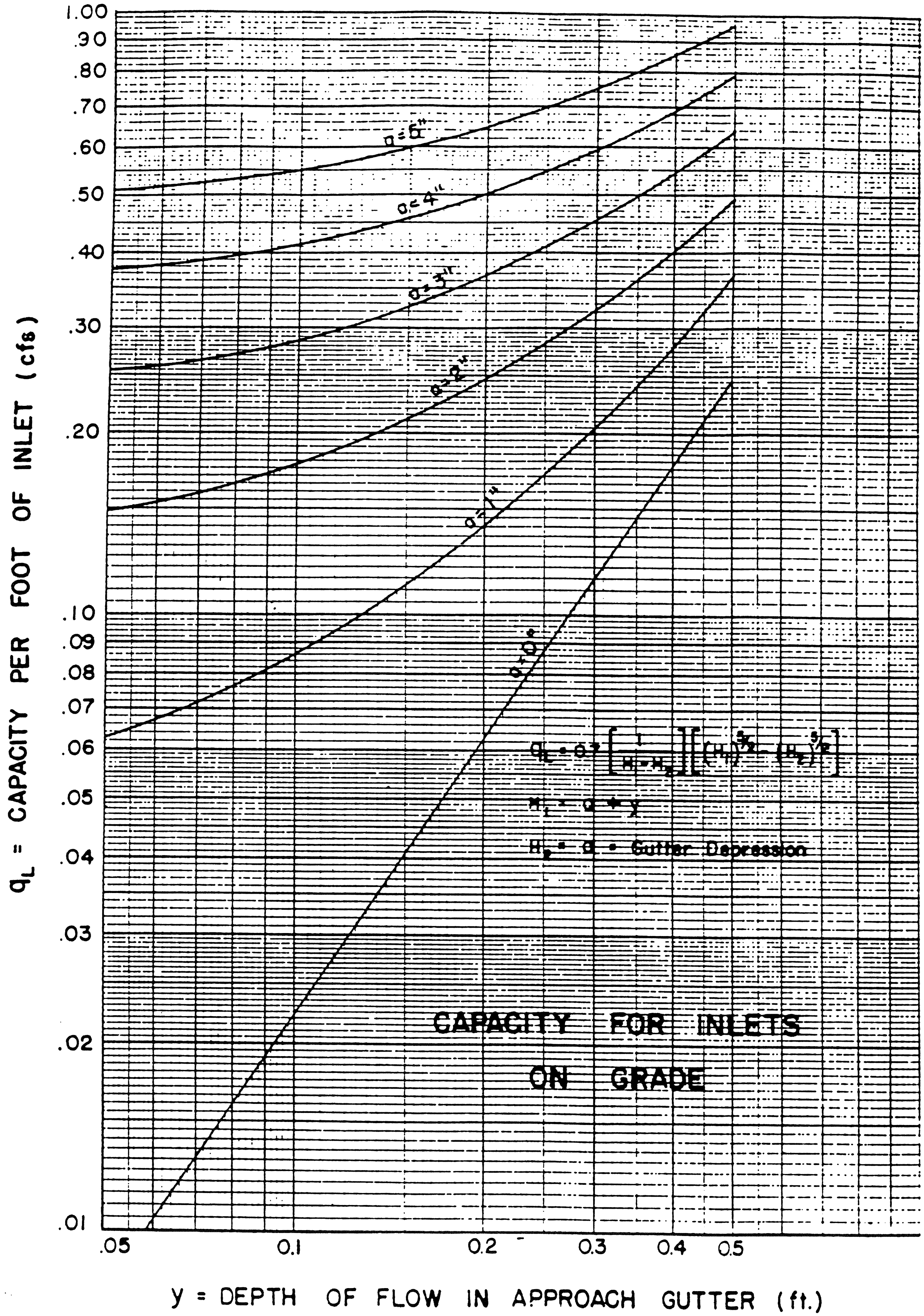


Figure 4-3



CAPACITY FOR INLETS ON GRADE
q_L = CAPACITY PER FOOT OF INLET (cfs)
y = DEPTH OF FLOW IN APPROACH GUTTER (ft.)
1.00
.90
.80
.70
.60
.50
.40
.30
.20
.10
.09
.08
.07
.06
.05
.04
.03
.02
.01
.05
0.1
0.2
0.3
0.4
0.5
a=6"
a=4"
a=3"
a=2"
a=1"
a=0"
$q_L = 0.7\left[\frac{1}{H_1 - H_2}\right]\left[(H_1)^{5/2} - (H_2)^{5/2}\right]$
$H_1 = a + y$
$H_2 = a$ = Gutter Depression

Figure 4-4

5.0 FLOW IN STORM DRAINS AND THEIR APPURTENANCES

5.1 GENERAL

A general description of storm drainage systems and quantities of storm runoff is contained in Section 2.0 of this manual. It is the purpose of this section to consider the significance of the hydraulic elements of storm drains and their appurtenances to a storm drainage system.

Hydraulically, storm drainage systems are conduits (open or enclosed) in which unsteady and non-uniform free flow exists. Accordingly, storm drains are designed for open-channel flow to satisfy as well as possible the requirements for unsteady and non-uniform flow. Steady flow conditions may or may not be uniform.

Reinforced concrete pipe shall be used in all storm sewer systems maintained by the Town of Addison.

5.2 VELOCITIES AND GRADES

5.2.1 Minimum Grades

Storm drains should operate with velocities of flow sufficient to prevent excessive deposition of solid materials; otherwise, objectionable clogging may result. The controlling velocity is near the bottom of the conduit and considerably less than the mean velocity. Storm drains shall be designed to have a minimum mean velocity of 3.0 fps, flowing full. Table 5-1 indicates the grades for concrete pipe (n=0.013) and for corrugated metal pipe (n=0.024) required to produce a velocity of 3.0 fps, which is considered to be the lower limit of scouring velocity. The minimum pipe slope used for standard construction procedures shall be 0.45%, unless otherwise approved by the Town Engineer.

TABLE 5-1

MINIMUM SLOPE REQUIRED TO PRODUCE SCOURING VELOCITY

Pipe Size (inches)	Concrete Pipe Slope (ft/ft)	Corrugated Metal Pipe Slope (ft/ft)
18	0.0018	0.0060
21	0.0015	0.0049
24	0.0013	0.0041
27	0.0011	0.0035
30	0.0009	0.0031
36	0.0007	0.0024
42	0.0006	0.0020
48	0.0005	0.0016
54	0.0004	0.0014
60	0.0004	0.0012
66	0.0004	0.0011
72	0.0003	0.0010
78	0.0003	0.0009
84	0.0003	0.0008
96	0.0002	0.0007

Source: City of Waco, Texas Storm Drainage Design Manual.

Note: Corrugated Metal Pipe is not to be used in systems maintained by the Town of Addison.

5.2.2 Maximum Velocities

Maximum velocities in conduits are important mainly because of the possibility of excessive erosion on the storm drain inverts. Table 5-2 shows the limits of maximum velocity for storm drains.

5.2.3 Minimum Diameter

Pipes which are to become an integral part of the public storm sewer system shall have a minimum diameter of 18 inches.

5.3 MATERIALS

In selecting a roughness coefficient for concrete pipe, between 0.011 and 0.015, consideration shall be given to the average conditions during the useful life of the structure. An "n" value of 0.017 for concrete pipe shall be used primarily in analyzing old conduits where alignment is poor and joints have become rough. If, for example, concrete pipe is being designed at a location where there is reason to believe that the roughness would increase through erosion or corrosion of the interior surface, slight displacement of joints or entrance of foreign materials, a roughness coefficient will be selected which, in the judgment of the designer, will represent the average condition. Any selection of "n" values below the minimum or above the maximum, for monolithic concrete structures, concrete pipe or corrugated metal pipe, must have the written approval of the Town Engineer.

The coefficients of roughness listed in Table 5-3 are for use in the nomographs contained herein, or for direct solution of Manning's Equation.

TABLE 5-2

MAXIMUM VELOCITY IN STORM DRAINS

Description	Maximum Permissible Velocity
Culverts (all types)	15 fps
Storm Drains (inlet laterals)	No Limit
Storm Drains (collectors)	15 fps
Storm Drains (mains)	12 fps

Source: City of Waco, Texas Storm Drainage Design Manual.

TABLE 5-3

ROUGHNESS COEFFICIENTS ("n") FOR STORM DRAINS

Materials of Construction	Recommended Design Coefficient	Range of Manning Coefficient
Concrete Pipe	0.012	0.011-0.015
Corrugated Metal Pipe		
Plain or Coated	0.024	0.022-0.026
Paved Invert	0.020	0.018-0.022

5.4 FULL OR PART FULL FLOW IN STORM DRAINS

5.4.1 General

All storm drains shall be designed by the application of the Continuity Equation and Manning's Equation either through the appropriate charts and nomographs or by direct solutions of the equations as follows:

$$Q = AV$$

and

$$Q = \frac{1.49}{n} AR^{2/3} S_f^{1/2}$$

where:

	Q	=	pipe flow (cfs);
	A	=	cross-sectional area of pipe (ft^2);
	V	=	velocity of flow (fps);
	n	=	coefficient of roughness of pipe;
	R	=	hydraulic radius = A/W_p (ft);
	S_f	=	friction slope in pipe (ft/ft);
	W_p	=	wetted perimeter.

There are several general rules to be observed when designing storm sewer runs. When followed, they will tend to alleviate or eliminate the common mistakes made in storm sewer design. These rules are as follows:

1. Select a pipe size and slope so that the velocity of flow will increase progressively, or at least will not appreciably decrease, at inlets, bends or other changes in geometry or configuration.

2. Do not discharge the contents of a larger pipe into a smaller one, even though the capacity of the smaller pipe may be greater due to a steeper slope.

3. At changes in pipe size, match the soffits of the two pipes at the same level rather than matching the flow lines.

4. Conduits are to be checked at the time of their design with reference to critical slope. If the slope of the line is greater than critical slope, the unit will likely be operating under entrance control instead of the originally assumed normal flow. Conduit slope should be kept below critical slope if at all possible. This also removes the possibility of a hydraulic jump within the line.

5. Where laterals tie to trunk lines, the laterals will be made at a 60° angle with the trunk line and the connection will be where the longitudinal centers intersect.

5.4.2 Pipe Flow Charts

Figures 5-1 through 5-3 are nomographs for determining flow properties in circular pipe. The nomographs are based upon a value of "n" of 0.012 for concrete and 0.024 for corrugated metal. The charts are self-explanatory, and their use is demonstrated by the example in Figure 5-1.

For values of "n" other than 0.012, the value of Q should be modified by using the formula below:

$$Q_c = \frac{Q_n(0.012)}{n_c}$$

where:

	Q_c	=	flow based upon n_c;
	n_c	=	value of "n" other than 0.012;
	Q_n	=	flow from the nomograph based on n = 0.012

This formula is used in two ways. If n_c=0.015 and Q_c is unknown, use the known properties to find Q_n from the nomograph, and then use the formula to convert Q_n to the required Q_c. If Q_c is one of the known properties, you must use the formula to convert Q_c (based on n_c) to Q_n (based on n=0.012) first, and then use Q_n and the other known properties to find the unknown value on the nomograph.

Example

Given: Slope = 0.005, depth of flow (d) = 1.8, diameter D = 36", n = 0.018

Find: Discharge (Q)

First determine d/D = 1.8'/3.0' = 0.6. Then enter Figure 5-1 to read Q_n=34 cfs. Using the formula, Q_c = 34 (0.012/0.018) = 22.7 cfs (Answer).

5.5 HYDRAULIC GRADIENT AND PROFILE OF STORM DRAINS

In storm drain systems flowing full, all losses of energy through resistance to flow in pipes, by changes of momentum or by interference with flow patterns at junctions, must be accounted for by accumulating head losses along the system from its initial upstream inlet to its outlet. The purpose of the determination of head losses at junctions is to include these values in a progressive calculation of the hydraulic gradient along the storm drain system. In this way, it is possible to determine the water surface elevation which will exist at each structure.

Following the computation of the quantity of storm runoff entering each inlet, the size and gradient of the pipe required to carry the runoff must determined. The following criteria provide the starting elevation of the hydraulic gradient at the outfall of the system:

1. The 100-year water surface elevation in a creek, stream or other open channel is to be calculated for the time of peak pipe discharge in the same storm.

2. For pipes that flow into sumps, ponds or other retention facilities, the hydraulic gradient should start at the 100-year design flood elevation in the sump.

3. When a proposed storm sewer is to be connected to an existing storm sewer system that has a flow greater than the proposed, the hydraulic gradient for the proposed storm sewer should start at the elevation of the existing storm sewer's hydraulic gradient.

4. No connection to an existing storm sewer shall be allowed where the existing system has insufficient capacity to carry the additional flow from the proposed storm sewer.

The friction head loss along the pipe shall be determined by direct application of Manning's Equation or by the appropriate nomographs in this section. The friction head loss over a given length of pipe may be calculated directly from the following equations:

$$h_f = S_f\, L$$

$$S_f = \left[\frac{Qn}{1.486AR^{2/3}}\right]^2$$

where:

h_f	=	friction head loss;
S_f	=	friction slope;
L	=	length of the pipe;
Q	=	100-year design flow in the pipe.

Minor losses due to turbulence at structures shall be determined by the procedure outlined in Section 5.8. The hydraulic grade line shall in no case be closer to the surface of the ground or street than 1.5 feet unless otherwise approved by the Town Engineer. If the storm sewer system is to be extended at some future date, present and future operation of the system must be considered.

5.6 MANHOLE LOCATION

Manholes shall be located at intervals not to exceed 600 feet for pipe 30 inches in diameter or smaller. Manholes shall preferably be located at street intersections, conduit junctions, changes of grade or changes of alignment.

Manholes for pipe greater than 30 inches in diameter shall be located at points where design indicates entrance into the conduit is desirable; however, in no case shall the distance between openings or entrances be greater than 1,200 feet.

5.7 PIPE CONNECTIONS

Prefabricated wye connections are available up to and including 24" x 24". Connections larger than 24 inches will be made by field connections. This recommendation is based primarily on the fact that field connections are more easily fitted to a given alignment than are precast connections. Regardless of the amount of care exercised by the contractor in laying the pipe, gain in footage invariably throws precast connections slightly out of alignment. This error increases in magnitude as the size of the pipe increases.

5.8 MINOR HEAD LOSSES AT STRUCTURES

The following total energy head losses at structures shall be determined for inlets, manholes, wye branches or bends in the design of closed conduits. See Table 5-4 and Figure 5-10 for details of each case. The minimum head loss used at any structure shall be 0.10 foot, unless otherwise approved.

The basic equation for most cases, where there is both upstreasm and downstream velocity, takes the form as set forth below:

$$h_l = K_l \frac{V_2^2 - V_1^2}{2g}$$

TABLE 5-4

JUNCTION OR STRUCTURE COEFFICIENT OF LOSS

Case No.	Reference Figure	Description of Condition	Coefficient K_j
I	5-4	Inlet on Main Line*	0.50
II	5-4	Inlet on Main Line with Branch Lateral*	0.25
III	5-4	Manhole on Main Line with 45° Branch Lateral	0.50
IV	5-4	Manhole on Main Line with 90° Branch Lateral	0.25
V	5-4	45° Wye Connection or Cut-in	0.75
VI	5-4	Inlet or Manhole at Beginning of Line	1.25
VII	5-4	Conduit on Curves for 90°**	
		Curve radius = diameter	0.50
		Curve radius = (2 to 8) diameter	0.40
		Curve radius = (8 to 20) diameter)	0.25
VIII	5-4	Bends Where Radius is Equal to Diameter	
		90° Bend	0.50
		60° Bend	0.48
		45° Bend	0.35
		22½° Bend	0.20
		Manhole on Line with 60° Lateral	0.35
		Manhole on Line with 22½° Lateral	0.75

* Must be approved by Town Engineer.
** Where bends other than 90° are used, the 90° bend coefficient can be used with the following percentage factors applied:
60° Bend - 85%; 45° Bend - 70%; 22½° Bend - 40%

Source: City of Waco, Texas Storm Drainage Design Manual

where: h_l = junction or structure head loss in feet;
V_l = velocity in upstream pipe in fps;
V_2 = velocity in downstream pipe in fps;
K_l = junction or structure coefficient of loss.

In the case where the initial velocity is negligible, the equation for head loss becomes:

$$h_g = K_j \frac{V_2^2}{2g}$$

At wye connections and field connections with a 60° angle of intersection, or at pipe size changes, the value of K_j shall be as follows:

Where $V_1 < V_2$ $\quad$ $Kj = 1.0$
Where $V_1 > V_2$ $\quad$ $Kj = 0.5$

5.9 UTILITIES

In the design of a storm drainage system, the engineer is frequently confronted with the problem of grade conflict between the proposed storm drain and existing utilities such as water, sanitary sewer, gas, telephone, electricity and cable television lines.

When conflicts arise between a proposed drainage system and a utility system, the owner of the utility system shall be contacted and made aware of the conflict. Any adjustments necessary to either the drainage system or the utility can then be determined. The design engineer is encouraged to contact the appropriate utility companies early in the design process.

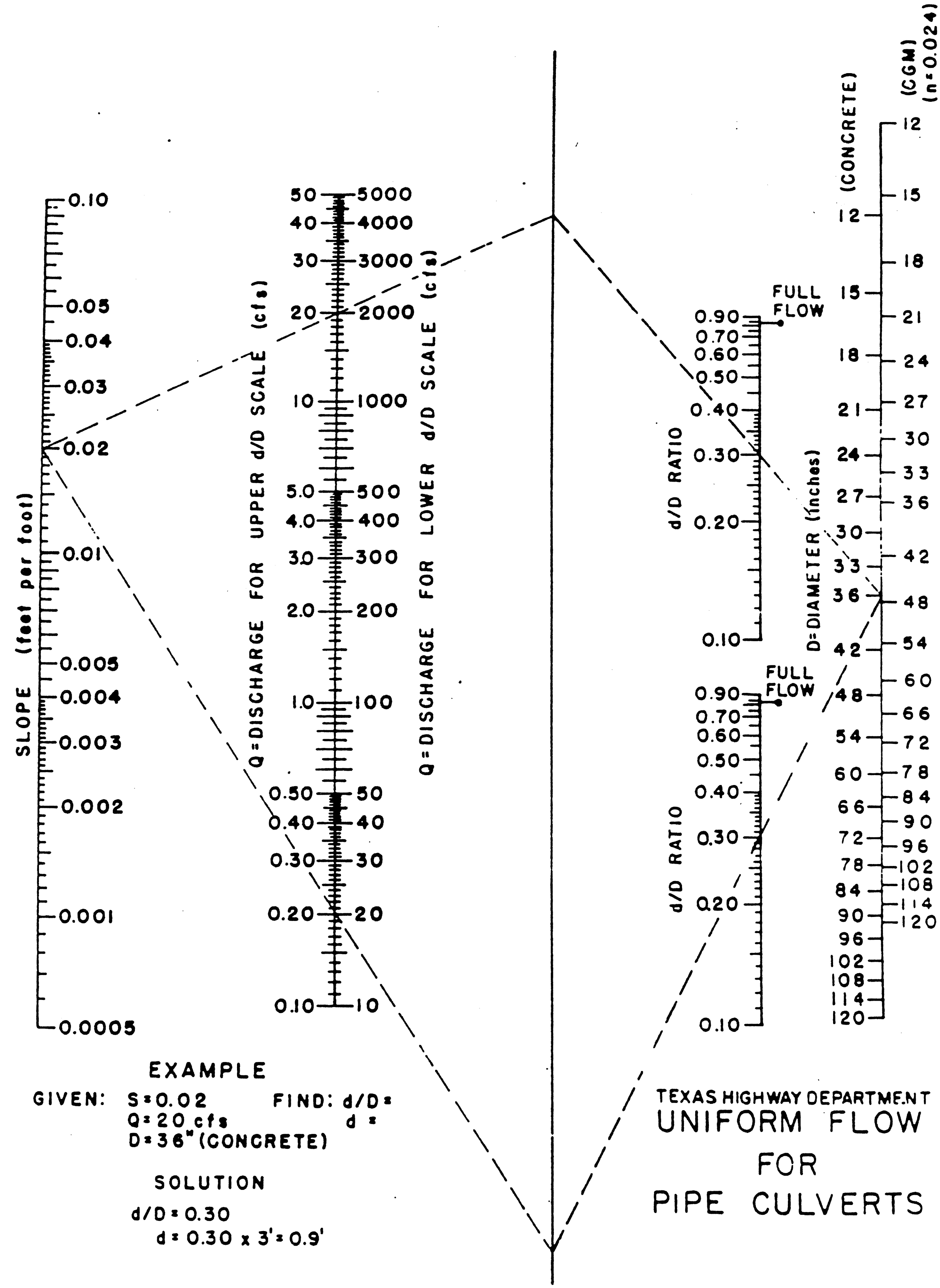
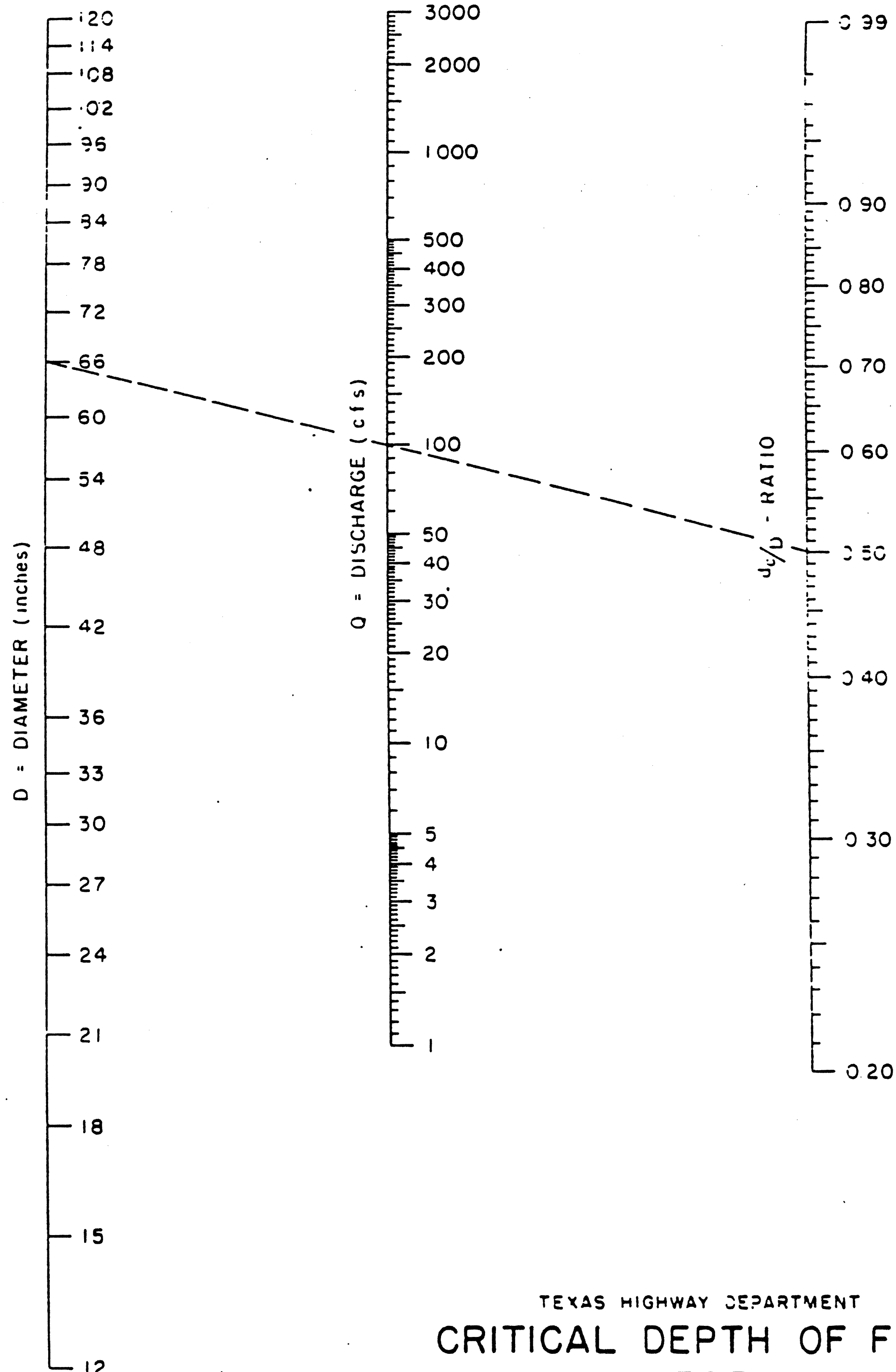

SLOPE (feet per foot)
0.10
0.05
0.04
0.03
0.02
0.01
0.005
0.004
0.003
0.002
0.001
0.0005
Q=DISCHARGE FOR UPPER d/D SCALE (cfs)
Q=DISCHARGE FOR LOWER d/D SCALE (cfs)
50
40
30
20
10
5.0
4.0
3.0
2.0
1.0
0.50
0.40
0.30
0.20
0.10
5000
4000
3000
2000
1000
500
400
300
200
100
50
40
30
20
10
d/D RATIO
FULL FLOW
0.90
0.70
0.60
0.50
0.40
0.30
0.20
0.10
D=DIAMETER (inches)
(CONCRETE)
(CGM) (n=0.024)
12
15
18
21
24
27
30
33
36
42
48
54
60
66
72
78
84
90
96
102
108
114
120
EXAMPLE
GIVEN: S=0.02 FIND: d/D=
Q=20 cfs d=
D=36" (CONCRETE)
SOLUTION
d/D=0.30
d=0.30 x 3'=0.9'
TEXAS HIGHWAY DEPARTMENT
UNIFORM FLOW
FOR
PIPE CULVERTS

Figure 5-1



TEXAS HIGHWAY DEPARTMENT

CRITICAL DEPTH OF FLOW FOR CIRCULAR CONDUITS

Figure 5-2

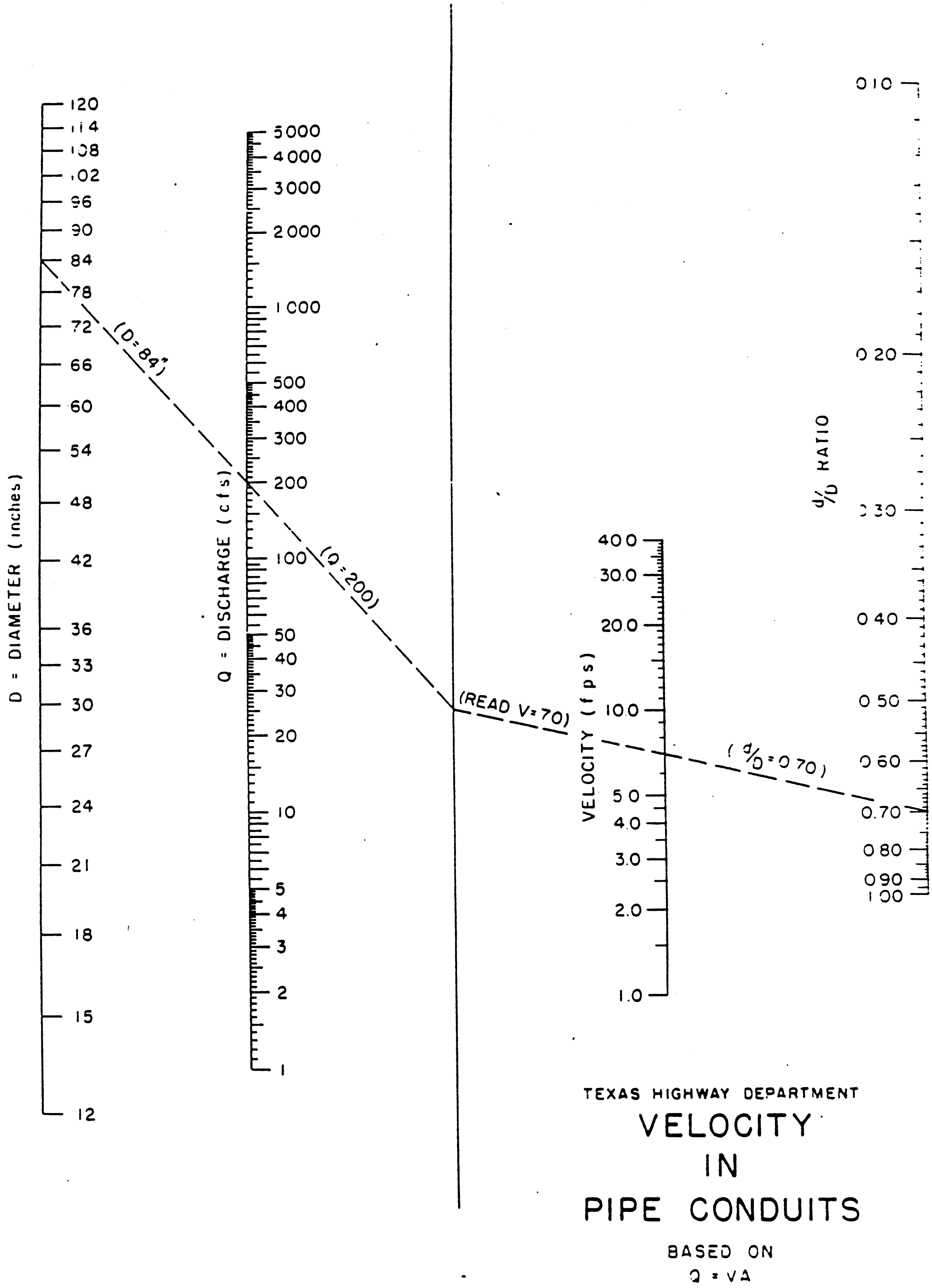
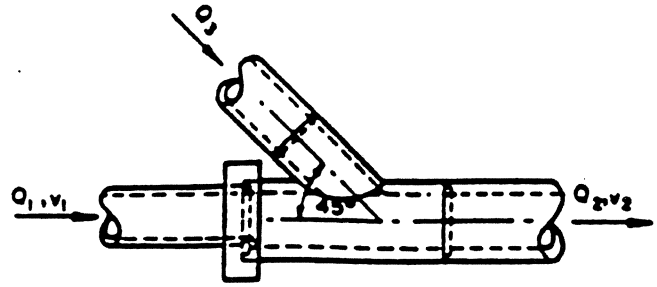
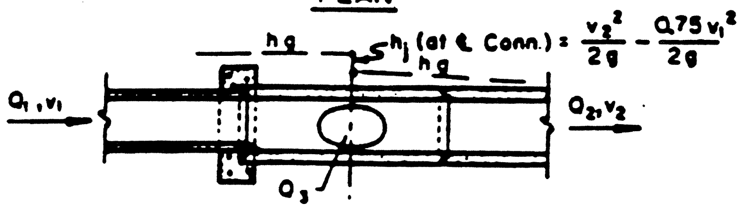

D = DIAMETER (inches)
Q = DISCHARGE (cfs)
VELOCITY (fps)
d/D RATIO
(D=84")
(Q=200)
(READ V=7.0)
(d/D=0.70)
TEXAS HIGHWAY DEPARTMENT
VELOCITY
IN
PIPE CONDUITS
BASED ON
Q = VA

Figure 5-3

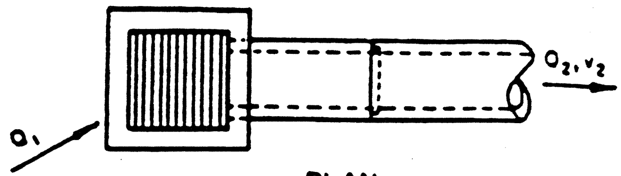


PLAN

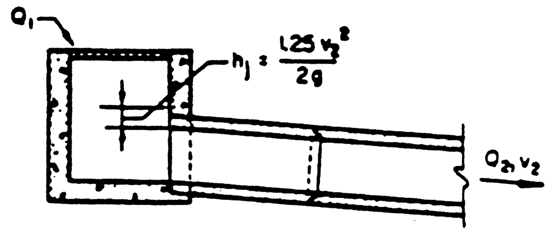


SECTION

CASE V
45° WYE CONNECTION OR CUT IN

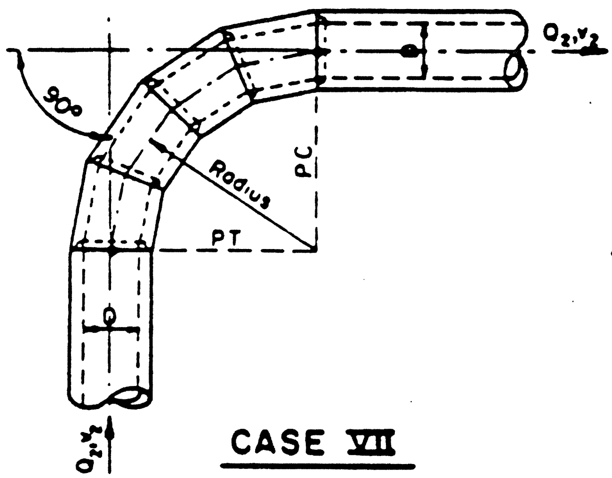


PLAN



SECTION

CASE VI
INLET OR MANHOLE AT BEGINNING OF LINE



CASE VII
CONDUIT ON 90° CURVES*

NOTE: Head loss applied at P.C. for length of curve.

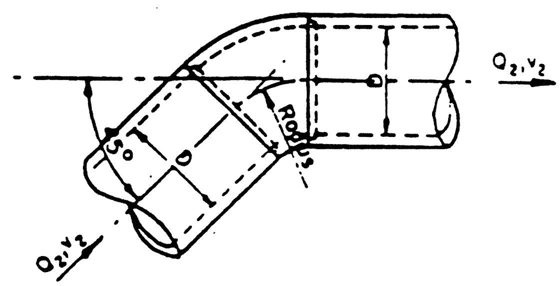
Radius = Dia. of Pipe $h_j = 0.50 \frac{v_2^2}{2g}$

Radius = (2-8) Dia of Pipe $h_j = 0.25 \frac{v_2^2}{2g}$

Radius = (8-20) Dia of Pipe $h_j = 0.40 \frac{v_2^2}{2g}$

Radius = Greater than 20 Dia. of Pipe $h_j = 0$

*When curves other than 90° are used, apply the following factors to 90° curves.
60° curve 85%
45° curve 70%
22 ½° curve 40%



CASE VIII
BENDS WHERE RADIUS IS EQUAL TO DIAMETER OF PIPE

NOTE: Head loss applied at begining of bend

90° Bend $h_j = 0.50 \frac{v_2^2}{2g}$

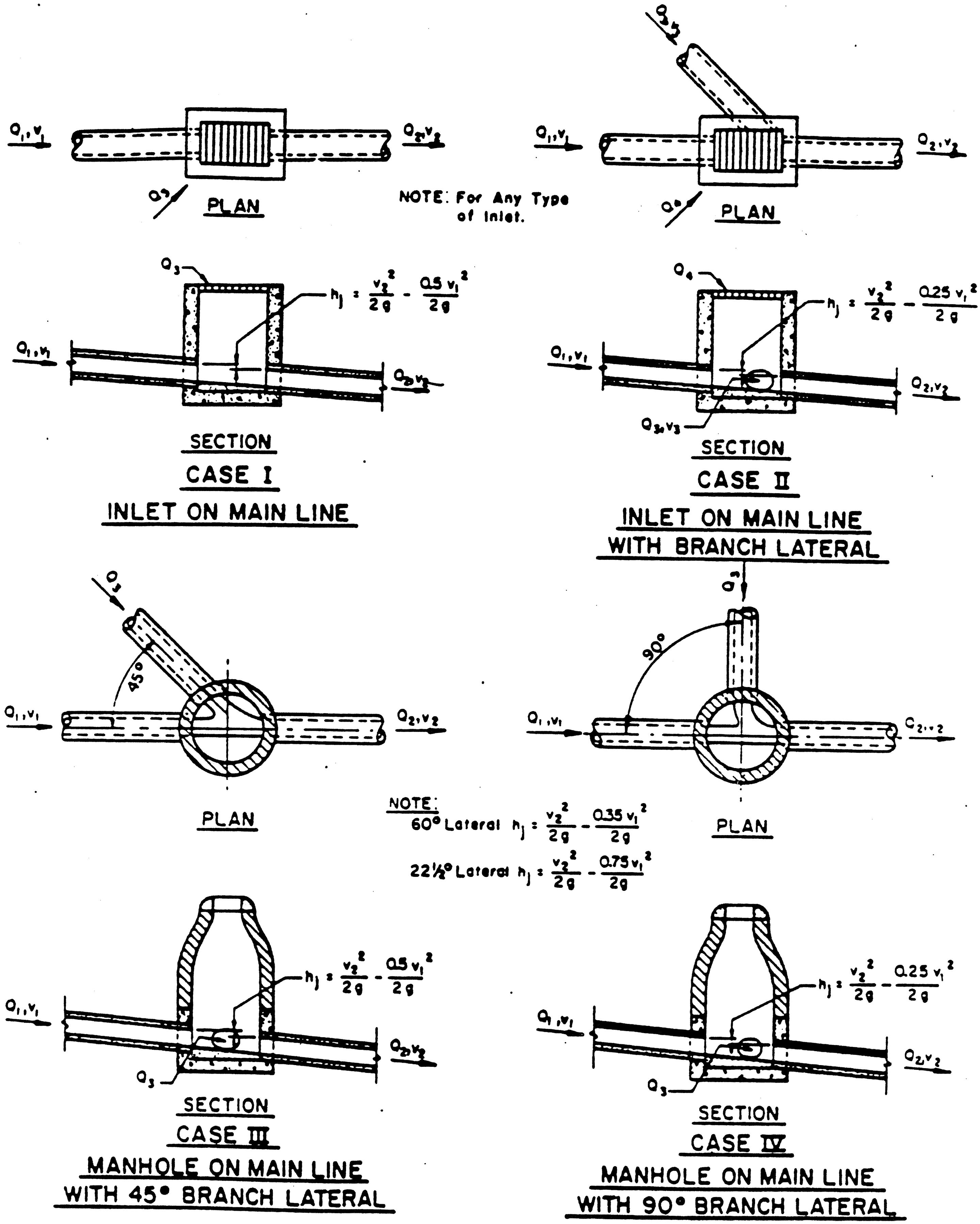
60° Bend $h_j = 0.43 \frac{v_2^2}{2g}$

45° Bend $h_j = 0.35 \frac{v_2^2}{2g}$

22 ½° Bend $h_j = 0.20 \frac{v_2^2}{2g}$

MINOR HEAD LOSSES DUE TO TURBULENCE AT STRUCTURES

Figure 5-4



MINOR HEAD LOSSES DUE TO TURBULENCE AT STRUCTURES

Figure 5-4

6.0 DESIGN OF ENCLOSED STORM DRAINAGE SYSTEMS

6.1 GENERAL

All storm drains shall be designed by the application of the Manning Equation either directly or through the appropriate charts or nomographs. In the preparation of hydraulic designs, a thorough investigation shall be made of all existing structures and their performance on the waterway in question.

The design of a storm drainage system should be governed by the following six conditions:

1. The system must accommodate all surface runoff resulting from the 100-year design storm without serious damage to physical facilities or substantial interruption of normal traffic.

2. Runoff resulting from storms exceeding the 100-year design storm must be anticipated and disposed of with minimum damage to physical facilities and minimum interruption of normal traffic.

3. The storm drainage system must have a maximum reliability of operation.

4. The storm drainage system shall be designed so that construction may proceed in an efficient and orderly manner, with minimum disruption of existing facilities, including streets and utilities.

5. The storm drainage system must require minimum maintenance and must be accessible for maintenance operations.

6. The storm drainage system must be adaptable to future expansion with minimum additional cost.

An example of the design of a storm drainage system is outlined in Sections 6.3 and 6.4. The design theory has been presented in the preceding sections.

6.2 PRELIMINARY DESIGN CONSIDERATIONS

Preliminary design considerations are as follows:

1. Prepare a drainage map of the entire area to be drained by the proposed improvements in accordance with the requirements of Section 9.3. Contour maps may provide topographic information for the drainage area map when supplemented by field reconnaissance.

2. Locate existing storm drainage facilities which may impact the project design.

3. Make a tentative layout of the proposed storm drainage system, locating all inlets, manholes, mains, laterals, ditches, culverts, etc.

4. Outline the drainage area for each inlet in accordance with present and future street development.

5. Locate all existing underground utilities.

6. Establish the minimum inlet time of concentration.

7. Establish the typical cross-section of each street.

8. Establish the permissible spread of water on all streets within the drainage area.

6.3 INLET SYSTEM

Determining the size and location of inlets is largely a trial-and-error procedure. Using the criteria outlined in Sections 2.0, 3.0 and 4.0 of this manual, the following steps will serve as a guide to the procedure to be used.

1. Beginning at the upstream end of the project drainage basin, outline a trial subarea and calculate the runoff from it.

2. Compare the calculated runoff to the allowable street capacity. If the calculated runoff is greater than the allowable street capacity, reduce the size of the trial subarea. If the calculated runoff is less than street capacity, increase the size of the trial subarea. Repeat this procedure until the calculated runoff equals the allowable street capacity. This is the first point at which a portion of the flow must be removed from the street. Note that other features such as street intersections may require that inlets be located before the allowable street capacity is reached. When selecting the percentage of flow to be removed by an inlet and, consequently, the size of the inlet, the design engineer shall compare the street capacity with the additional runoff entering the street downstream of the inlet.

3. Record the inlet size, the amount of intercepted flow and amount of flow carried over (bypassing the inlet).

4. Continue the above procedure for other subareas until a complete system of inlets has been established. Remember to account for carryover from one inlet to the next.

5. After a complete system of inlets has been established, modification should be made to accommodate special situations such as point sources of large quantities of runoff, and variation of street alignments and grades.

6. After the inlets have been located and sized, the inlet pipes can be designed.

7. Inlet pipes are sized to carry the volume of water intercepted by the inlet. Inlet pipe capacities may be controlled by the pipe gradient available, or by entry conditions into the pipe at the inlet. Inlet pipe sizes should be determined for both conditions and the larger size used.

6.4 STORM SEWER SYSTEM

After computation of the quantity of storm runoff entering each inlet, the storm sewer system required to carry the accumulated runoff is designed. It should be borne in mind that the quantity of flow to be carried by any particular section of the storm sewer system is not the sum of the inlet design quantities of all inlets above that section of the system, but is less than the straight total. This situation results from an increasing time of concentration as runoff passes along the storm sewer system, causing a corresponding decrease in the rainfall intensity.

6.4.1 Storm Sewer Pipe

The proposed ground line profile is now used in conjunction with the previous runoff calculations. The elevation of hydraulic gradient is arbitrarily established approximately 2 feet below the proposed ground surface. When this tentative gradient is set and the design discharge is determined, a Manning flow chart may be used to determine the pipe size and velocity.

It is probable that the tentative gradient will have to be adjusted at this point since the intersection of the discharge and the slope on the chart will likely occur between standard pipe sizes. Velocities can be read directly from a Manning Flow Chart based on a given discharge, pipe size and gradient slope.

Once pipe sizes have been selected for the storm sewer system, the hydraulic gradient must be recalculated to incorporate head losses which occur within the system. At each point where an inlet pipe connects to the main pipe, the hydraulic gradient of the inlet pipe must be checked to ensure that it remains below the gutter grade at the inlet. The hydraulic gradient should be plotted directly on the construction plan profile.

6.4.2 Pipe Size Changes, Junctions, Inlets and Manholes

1. Head losses must be calculated and incorporated in the hydraulic gradient for all pipe size changes, junctions, inlets, manholes or other features which change the flow characteristics within the storm sewer system.

2. Determine the velocity of flow for the incoming pipe (main line) at the pipe size change, junction, inlet or manhole.

3. Determine the velocity of flow for the outgoing pipe (main line) at the pipe size change, junction, inlet or manhole.

4. Compute the velocity head for the incoming velocity (main line).

5. Compute the velocity head for the outgoing velocity (main line).

6. Determine the head loss coefficient K_l at the design point from Table 5-4 or Figure 5-4.

7. Compute the head loss at the pipe size change, junction, inlet or manhole, using the following formula:

$$h_l = K_l \left(\frac{[V_2]^2 - [V_1]^2}{2g}\right)$$

8. Compute the hydraulic gradient at the upstream end of the junction as if the junction were not there.

9. Add head loss to hydraulic gradient elevation determined to obtain hydraulic gradient elevation at the upstream end of the junction.

All information shall be recorded on the plans for convenient review.

6.4.3 Outlet Erosion Control

All storm sewer designs shall include a method of ensuring erosion control at the outlet. Methods may include outlet alignment, channel lining (concrete, gabions, rock riprap, etc.) near the outlet, concrete outlet slabs with sidewalls and stilling basins.

All information shall be recorded on the submitted plans or in tabular form convenient for review.

7.0 FLOW IN OPEN CHANNELS

7.1 GENERAL

The benefits of open channels as an alternative to closed storm sewer systems include lower construction cost, multiple use for recreational and aesthetic purposes, and potential for detention storage. The disadvantages include larger land requirements and higher maintenance costs. Careful planning and design are needed to minimize the disadvantages and to increase the benefits of open channels.

The ideal channel is a natural one carved by nature over a long period of time. The benefits of such a channel include the following:

1. Velocities are usually low, resulting in longer concentration times and lower downstream peak flows.

2. Channel storage tends to decrease peak flows.

3. Maintenance needs are usually low because the channel is somewhat stabilized.

4. The channel may provide a desirable greenbelt and recreational area, adding significant social benefits.

Channel stability is a well recognized problem in urban hydrology because of the significant increase in runoff which results from urban development. A natural channel must be studied to determine the measures needed to avoid future bottom scour and bank cutting. Erosion control measures shall be employed which protect the integrity of the channel while preserving the natural channel appearance wherever possible.

Generally speaking, the manmade channel which most nearly conforms to the character of a natural channel is the most desirable.

7.2 CHANNEL DISCHARGE

7.2.1 Manning's Equation

The hydraulic characteristics of channels shall be determined by Manning's equation:

$$Q = \frac{1.49}{n} AR^{2/3}S^{1/2}$$

where:	Q	=	total discharge in cfs;
	n	=	coefficient of roughness;
	A	=	cross-sectional area of channel in sq. ft.;
	R	=	hydraulic radius of channel in feet;
	S	=	slope of frictional gradient in feet/foot.

7.2.2 Uniform Flow

For a given channel condition of roughness, discharge and slope, there is only one possible depth for maintaining a uniform flow; this depth is the normal depth. When the roughness, depth and slope are known at a channel section, there can only be one discharge for maintaining a uniform flow through that section; this discharge is the normal discharge.

If the channel is uniform and resistance and gravity forces are in exact balance, the water surface will be parallel to the bottom of the channel. This is the condition of uniform flow.

Uniform flow is more often a theoretical abstraction than an actuality. True uniform flow is difficult to find in the field or to obtain in the laboratory. Channels are sometimes designed on the assumption that they will carry uniform flow at normal depths, but because of conditions which are difficult to evaluate, the flow will actually have depths considerably different from uniform depth. The engineer must be aware of the fact that the uniform flow computation provides only an approximation of what will occur; nevertheless, such computations are useful and necessary for planning.

7.2.3 Normal Depth

The normal depth is computed so frequently that it is convenient to use nomographs for various types of cross-sections to eliminate the need for trial-and-error solutions, which are time-consuming. A nomograph for uniform flow in trapezoidal channels in given in Figure 7-1.

7.3 WATER SURFACE PROFILES

Open channel flow in urban drainage systems is actually non-uniform because of changing channel sections and alignments, and the effect of culverts, bridges and other structures. This necessitates the use of backwater computations for all final channel design work.

A water surface profile must be computed for all channels and shown on the final plans. Computation of the water surface profile should utilize standard backwater methods or acceptable computer routines, taking into consideration all losses due to changes in velocity, drops, bridge openings and other restrictions.

7.4 DESIGN CONSIDERATIONS

In general, manmade channels should have a trapezoidal section of adequate cross-sectional area to take care of uncertainties in runoff estimates, changes in channel coefficients, channel obstructions and silt accumulation.

Accurate determination of the "n" value is critical in the analysis of the hydraulic characteristics of a channel. The "n" value for each channel reach should be based on experience and judgment with regard to the individual channel characteristics. Table 7-1 gives a method of determining the composite roughness coefficient based on actual channel conditions.

TABLE 7-1

COMPUTATION OF COMPOSITE ROUGHNESS COEFFICIENT FOR EXCAVATED AND NATURAL CHANNELS

$$n_c = (n_0 + n_1 + n_2 + n_3 + n_4)m$$

	Channel Conditions	Value
Material Involved n_o	Earth	0.020
	Rockcut	0.025
	Fine Gravel	0.024
	Coarse Gravel	0.028
Degree of Irregularity n_1	Smooth	0.000
	Minor	0.005
	Moderate	0.010
	Severe	0.020
Variation of Channel Cross-Section n_2	Gradual	0.000
	Alternating Occasionally	0.005
	Alternating Frequently	0.010-0.015
Relative Effect of Obstructions n_3	Negligible	0.000
	Minor	0.010-0.015
	Appreciable	0.020-0.030
	Severe	0.040-0.060
Vegetation n_4	Low	0.005-0.010
	Medium	0.010-0.025
	High	0.025-0.050
	Very High	0.050-0.100
Degree of Meandering m	Minor	1.000-1.200
	Appreciable	1.200-1.500
	Severe	1.500

Roughness Coefficient for Lined Channels
Concrete Lined - $n = 0.017$
Rubble Riprap - $n = 0.022$

Open Channel Hydraulics
Ven te Chow, Ph.D.

Where practicable, unlined channels should have sufficient gradient to provide velocities that will be self-cleaning, but not so great that erosion results. Lined channels, drop structures, check dams or concrete spillways may be required to control the erosion that results from large volumes of water flowing at high velocities. Unless approved otherwise by the Town Engineer, channel velocities in manmade channels shall not exceed 7 fps.

7.5 CHANNEL CROSS-SECTIONS

The channel shape may vary to better suit the particular location and environmental conditions. All channel geometrics are subject to review and approval by the Town Engineer.

7.5.1 Side Slope

Except in horizontal curves, flatter channel side slopes are preferred over steeper slopes. Unless approved by the Town Engineer, channel side slopes shall be no steeper than 3:1, which is also the practical limit for mowing equipment. Rock- or concrete-lined channels, or those which for other reasons do not require slope maintenance, may have slopes as steep as 1.5:1.

7.5.2 Depth

Deep channels are difficult to maintain and can be hazardous. Therefore, manmade channels should be as shallow as practical.

7.5.3 Bottom Width

Channels with narrow bottoms are difficult to maintain and are conducive to high velocities. It is desirable to design open channels such that the bottom width is at least twice the depth.

7.5.4 Pilot Channels

Low flows from urban areas may require special attention in the design of channels. If erosion of the channel bottom appears to be a likely problem, low flows shall be carried in a paved pilot channel which has a capacity of 5% of the design peak flow. Care must be taken to ensure that low flows are directed into the pilot channel without eroding out a parallel channel. Pilot channels should include adequate toewalls to prevent undermining. Erosion control mats are also recommended to allow faster growth of natural vegetation, seeding or hydromulching before erosion commences. Pilot channels shall also be designed to permit maintenance vehicle passage along the channel.

7.5.5 Freeboard

For channels which flow at higher velocities, the combined effects of surface roughness, wave action, air bulking and splash are quite erosive along the top of the flow. Freeboard must be provided between the top of the flow and the channel bank to prevent damage to adjacent improvements. Unless approved otherwise by the Town Engineer, all channels shall have a minimum height of freeboard of 1 foot based on the 100-year design storm.

7.6 STRUCTURES

The designer must exercise sound judgment in the design of hydraulic structures to ensure the long-term integrity of the structures while minimizing their maintenance requirements. The designer of hydraulic structures should refer to standards adopted by the U.S. Army Corps of Engineers and the Bureau of Reclamation, which are based on proven hydraulic models. Paved trapezoidal drop structures tend to require excessive maintenance and should be avoided. Erosion protection along and at each end of structures, along with adequate toewalls and foundation drainage, is mandatory. In addition, the use of hydraulic structures in the urban environment requires that consideration be given to the structures' appearance.

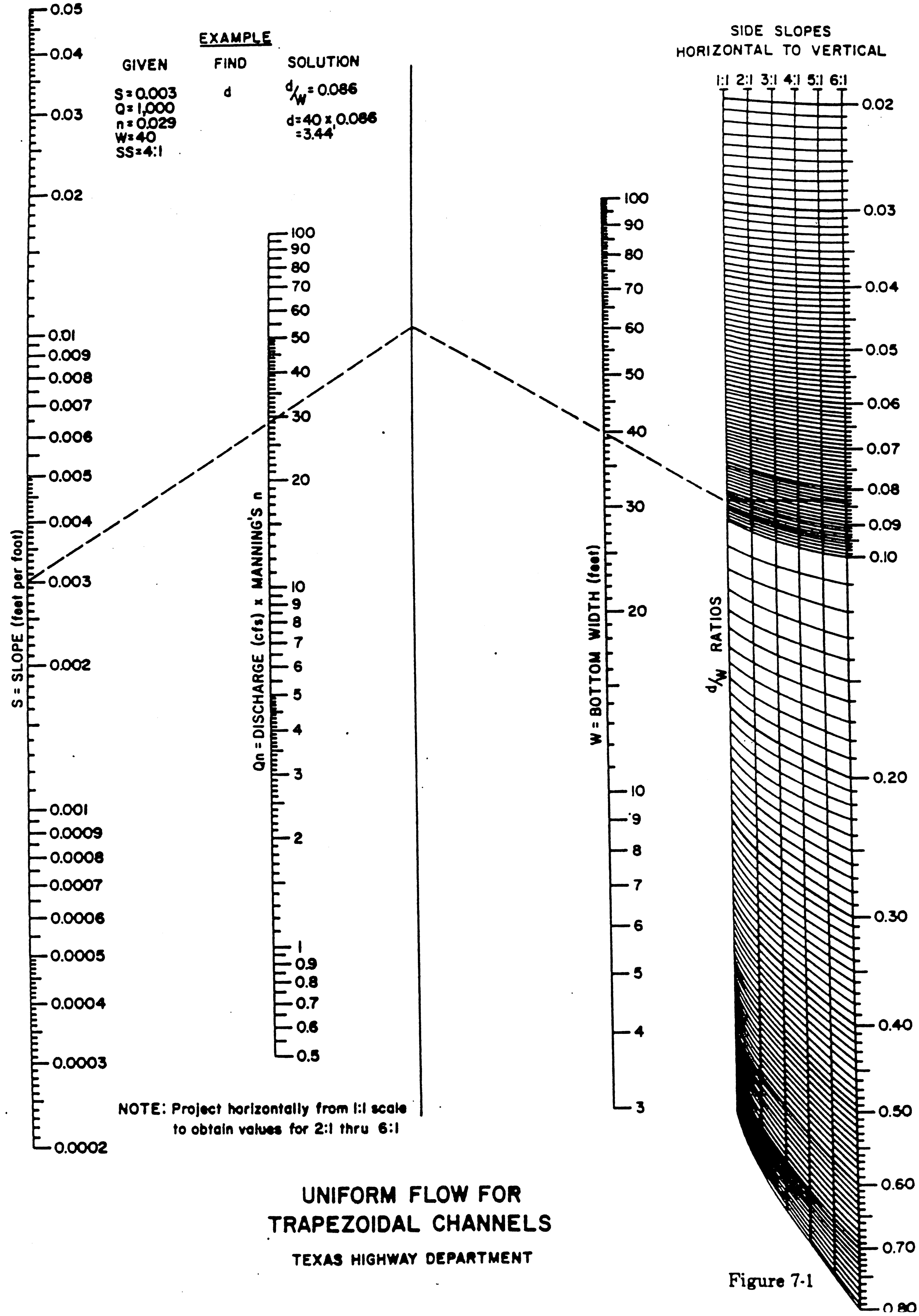

EXAMPLE
GIVEN
S=0.003
Q=1,000
n=0.029
W=40
SS=4:1
FIND
d
SOLUTION
d/W = 0.086
d=40 x 0.086
=3.44'
S = SLOPE (feet per foot)
0.05
0.04
0.03
0.02
0.01
0.009
0.008
0.007
0.006
0.005
0.004
0.003
0.002
0.001
0.0009
0.0008
0.0007
0.0006
0.0005
0.0004
0.0003
0.0002
Qn = DISCHARGE (cfs) x MANNING'S n
100
90
80
70
60
50
40
30
20
10
9
8
7
6
5
4
3
2
1
0.9
0.8
0.7
0.6
0.5
W = BOTTOM WIDTH (feet)
100
90
80
70
60
50
40
30
20
10
9
8
7
6
5
4
3
SIDE SLOPES
HORIZONTAL TO VERTICAL
1:1 2:1 3:1 4:1 5:1 6:1
d/W RATIOS
0.02
0.03
0.04
0.05
0.06
0.07
0.08
0.09
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
NOTE: Project horizontally from 1:1 scale
to obtain values for 2:1 thru 6:1
UNIFORM FLOW FOR
TRAPEZOIDAL CHANNELS
TEXAS HIGHWAY DEPARTMENT

Figure 7-1

7.7 RIGHT-OF-WAY REQUIRED FOR OPEN CHANNELS

The City Manager may require the dedication of drainageways and/or floodways for open channels, creeks and flumes to the Town of Addison. Rights-of-way shall encompass all areas having a ground elevation below the higher of 1 foot above the water surface elevation associated with the design flood or the top of the high bank, whichever provides the larger area. In all cases, the right-of-way shall also include at least a 15-foot wide maintenance strip along both sides of the channel or, if the Town Engineer so allows, at least a 20-foot wide maintenance strip along one side of the channel. Streets, alleys, bicycle paths, etc. alongside the channel can serve as all or part of the maintenance right-of-way.

Drainage right-of-way for flumes shall provide sufficient width to permit future maintenance accessibility, and in no case shall be less than 20 feet wide.

7.8 STREETS ALONG DRAINAGEWAYS

Where the Town of Addison has designated a floodway or floodplain as a dedicated drainageway or part of the Town park system, the following conditions may be required:

1. Parallel streets fronting along the park or dedicated drainageway.

2. Cul-de-sacs which provide public access fronting on the park.

3. Loop streets which provide public access fronting on the park.

In all cases, the City Manager shall have the right to review and approve the proposed street alignment fronting on Town parks, and shall in cases where the alignment is unsatisfactory, negotiate a satisfactory alignment with the developer and the Town Engineer.

8.0 DESIGN OF CULVERTS

8.1 GENERAL

The function of a drainage culvert is to pass the design storm flow under a roadway or railroad without causing excessive backwater and without creating excessive downstream velocities. The design engineer shall keep energy losses and discharge velocities within reasonable limits when selecting a structure which will meet these requirements. Reinforced concrete culverts shall be required for all culverts which will be maintained by the Town of Addison.

8.2 QUANTITY OF FLOW

The 100-year design storm shall be determined for culvert design calculations as outlined in Section 2.0 of this manual. In general, for culverts which carry flows under roadways, the freeboard between the upstream 100-year design water surface and the top of curb elevation shall be a minimum of 2 feet. This vertical distance provides a safety margin to protect against unusual clogging of the culvert, and further protects the street against future changes in the surrounding conditions.

For culverts which carry flow under facilities other than roadways, the designer shall provide an adequate freeboard to protect the facilities from flooding.

8.3 HEADWALLS AND ENDWALLS

The normal functions of properly designed headwalls and endwalls are to anchor the culvert to prevent movement due to lateral pressures, to control erosion and scour resulting from excessive velocities and turbulence, and to prevent adjacent soil from sloughing into the waterway opening. All headwalls shall be constructed of reinforced concrete and may be straight parallel headwalls, flared headwalls or warped headwalls with or without aprons as may be required by site conditions.

8.3.1 Conditions at Entrance

It is important to recognize that the operational characteristics of a culvert may be completely changed by the shape or conditions at the inlet or entrance. The design of culverts must include consideration of energy losses that may occur at the entrance. The entrance head losses may be determined by the following equation:

$$h_e = K_e \frac{(V_2)^2 - (V_1)^2}{2g}$$

where:

h_e	=	entrance head loss in feet;
K_e	=	entrance loss coefficient as shown in Table 8-1.
V_2	=	velocity of flow in the invert in fps;
V_1	=	velocity of approach in fps as shown in Table 8-2;

In order to compensate for the retarding effect on the approach velocity in channels produced by the creation of the headwater pools at the culvert entrance, the approach velocity in the channel (V_a) shall be reduced by the factors as shown in Table 8-2.

8.3.2 Type of Headwall or Endwall

In general, the following guidelines should be used in the selection of the type of headwall or endwall:

- Parallel Headwall and Endwall

  1. Approach velocities are low (below 6 fps).
  2. Backwater pools may be permitted.
  3. Approach channel is undefined.
  4. Ample right-of-way or easement is available.
  5. Downstream channel protection is not required.

TABLE 8-1

VALUES OF ENTRANCE LOSS COEFFICIENTS "K_e"

Type of Structure and Entrance Design	Value of K_e
Reinforced Concrete Box	
Submerged Entrance	
Parallel Wingwalls	0.5
Flared Wingwalls	0.4
Free Surface Flow	
Parallel Wingwalls	0.5
Flared Wingwalls	0.15
Concrete Pipe	
Projecting from Fill, Socket End	0.2
Projecting from Fill, Square Cut End	0.5
Headwall or Headwall and Wingwalls	
Socket End of Pipe	0.2
Square Edge	0.5
End-Section Conforming to Fill Slope	0.5
Corrugated Metal Pipe or Pipe-Arch	
Projecting from Fill (no headwall)	0.9
Headwall or Headwall and Wingwalls	
Square Edge	0.5
End-Section Conforming to Fill Slope	0.5

TABLE 8-2

REDUCTION FACTORS FOR VELOCITY OF APPROACH

Velocity of Approach V_a (fps)	Description of Conditions	V_1 to be Used in Formula for h_e
0 - 6	All culverts.	$V_1 = V_a$
Above 6	Good alignment of the approach channel; headwater pool permissible within the right-of-way.	$V_1 = 0.5V_a$
Above 6	Good alignment of the approach channel; channel slopes have been lined; limited backwater pool permissible within the right-of-way.	$V_1 = 0$

- Flared Headwall and Endwall
  1. Channel is well defined.
  2. Approach velocities are between 6 and 10 fps.
  3. Medium amounts of debris exist.

  The wings of flared headwalls should be located with respect to the direction of the approaching flow instead of the culvert axis.

- Warped Headwall and Endwall
  1. Channel is well defined and concrete lined.
  2. Approach velocities are between 8 and 20 fps.
  3. Medium amounts of debris exist.

  Warped headwalls are effective with drop-down aprons to accelerate flow through the culvert, and the endwalls are effective in transitioning flow from closed conduit flow to open channel flow at the outfall. This type of headwall should be used only where the drainage structure is large and the available right-of-way or easement is limited.

8.3.3 Special Culvert Designs

Culvert designs which involve improved inlet conditions such as side-tapered, slope-tapered or flared inlets must be submitted with design calculations to the Town Engineer. Improved inlets may be used to improve the hydraulic efficiency of culvert entrances for culverts which operate under inlet control.

Culvert cross-sections are typically circular or box-shaped; however, other culvert sections such as elliptical or pipe-arch may be considered by the Town Engineer provided that acceptable design calculations are provided with the construction plans.

8.4 CULVERT DISCHARGE VELOCITIES

The velocity of discharge from culverts should be limited as shown in Table 8-3. Consideration must be given to the effect of high velocities, eddies or other turbulence on the natural channel, downstream property and roadway embankments.

8.5 CULVERT FLOW CONTROLS

Generally, the hydraulic control in a culvert will be at the culvert outlet if the culvert is operating in a mild slope regime. Entrance control usually occurs if the culvert is operating in a steep slope regime.

For outlet control, the head losses due to the tailwater and barrel friction are predominant in controlling the headwater of the culvert. The entrance will allow the water to enter the culvert faster than the backwater effects of the tailwater and barrel friction will allow it to flow through the culvert.

For entrance control, the entrance characteristics of the culvert are such that the entrance head losses are predominant in determining the headwater of the culvert. The barrel will carry water through the culvert more efficiently than the water can enter the culvert.

Each culvert flow, however classified, is dependent upon one or both of these controls; therefore, because of the importance of these controls, further discussion follows.

8.6 SELECTION OF CULVERT SIZE AND TYPE

8.6.1 Culvert Operating Conditions

Culverts shall be selected based on hydraulic principles, economy of size and shape, and with a resulting headwater depth which will not cause damage to adjacent property. It is essential to the proper design of a culvert that the conditions under which the culvert will operate

TABLE 8-3

CULVERT DISCHARGE - VELOCITY LIMITATIONS

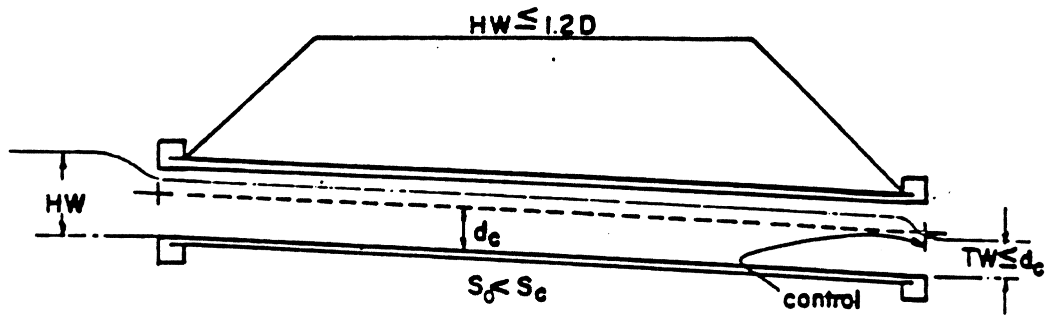
Downstream Condition	Maximum Allowable Discharge Velocity (fps)
Earth	6
Sod Earth	8
Shale	10
Rock	15
Paved or Riprap Apron	15

are known. Five types of operating conditions are listed below, with a discussion of each following.

Type I	Flowing Part Full with Outlet Control and Tailwater Depth Below Critical Depth
Type II	Flowing Part Full with Outlet Control and Tailwater Depth Above Critical Depth
Type IIIA	Flowing Part Full with Inlet Control
Type IIIB	Flowing Part Full with Inlet or Outlet Control
Type IVA	Flowing Full with Submerged Outlet
Type IVB	Flowing Full with Partially Submerged Outlet

Type I

Culvert Flowing Part Full with Outlet Control and Tailwater Depth Below Critical Depth



Conditions

The entrance is unsubmerged (HW≤1.2D), the slope at design discharge is sub-critical (S_o<S_c) and the tailwater is below critical depth (TW≤d_c).

The above condition is a common occurrence where the natural channels are on flat grades and have wide, flat floodplains. The control is critical depth at the outlet.

In culvert design, it is generally considered that the headwater pool maintains a constant level during the 100-year design storm. If this level does not submerge the culvert inlet, the culvert flows part full.

If critical flow occurs at the outlet, the culvert is said to have "Outlet Control." A culvert flowing part full with outlet control will require a depth of flow in the barrel of the culvert greater than critical depth while passing through critical depth at the outlet.

The capacity of a culvert flowing part full with outlet control and a tailwater depth below critical depth shall be governed by the following equation when the approach velocity is considered zero:

$$HW = d_c + \frac{(V_c)^2}{2g} + h_e + h_f - S_o L$$

where:

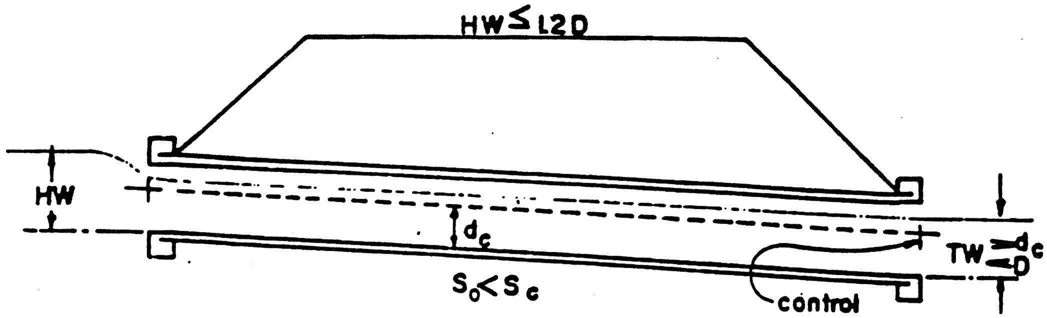
HW	=	Headwater depth above the invert of the upstream end of the culvert in feet. Headwater must be equal to or less than 1.2D or entrance is submerged and Type IV operation will result.
d_c	=	Critical depth of flow in feet.
	=	$\left[\frac{q^2}{32.2}\right]^{1/3}$
q	=	Discharge per foot of width for rectangular channels.
D	=	Diameter of pipe or height of box in feet.
V_c	=	Critical velocity in fps occurring at critical depth.
h_e	=	Entrance head loss in feet.
	=	$K_e \frac{V_c^2}{2g}$
K_e	=	Entrance loss coefficient (see Table 8-1).
h_f	=	Friction head loss in feet = $S_f L$.
S_f	=	Friction slope or slope that will produce uniform flow. For Type I operation, the friction slope is based upon $1.1d_c$. (See Figures 8-2 and 8-5.)

S_o = Slope of culvert in feet/foot.

L = Length of culvert in feet.

Type II

Culvert Flowing Part Full with Outlet Control and Tailwater Depth Above Critical Depth



Conditions

The entrance is unsubmerged (HW≤1.2D), the slope at design discharge is sub-critical ($S_o<S_c$) and the tailwater is above critical depth (TW>d_c).

The above condition is a common occurrence where the channel is deep, narrow and well-defined.

If the headwater pool elevation does not submerge the culvert inlet, the slope at design discharge is sub-critical, and the tailwater depth is above critical depth, the control is said to occur at the outlet, and the capacity of the culvert shall be governed by the following equation when the approach velocity is considered zero:

$$HW = TW + \frac{(V_{TW})^2}{2g} + h_e + h_f - S_oL$$

where: HW = Headwater depth above the invert of the upstream end of the culvert in feet. Headwater depth must be equal to or less than 1.2D or entrance is submerged and Type IV operation will result.

TW = Tailwater depth above the invert of the downstream end of the culvert in feet.

V_{TW} = Culvert discharge velocity in fps at tailwater depth.

h_e = Entrance head loss in feet.

$$= K_e \frac{V_{TW}^2}{2g}$$

K_e = Entrance loss coefficient (see Table 8-1).

h_f = Friction head loss in feet = $S_f L$.

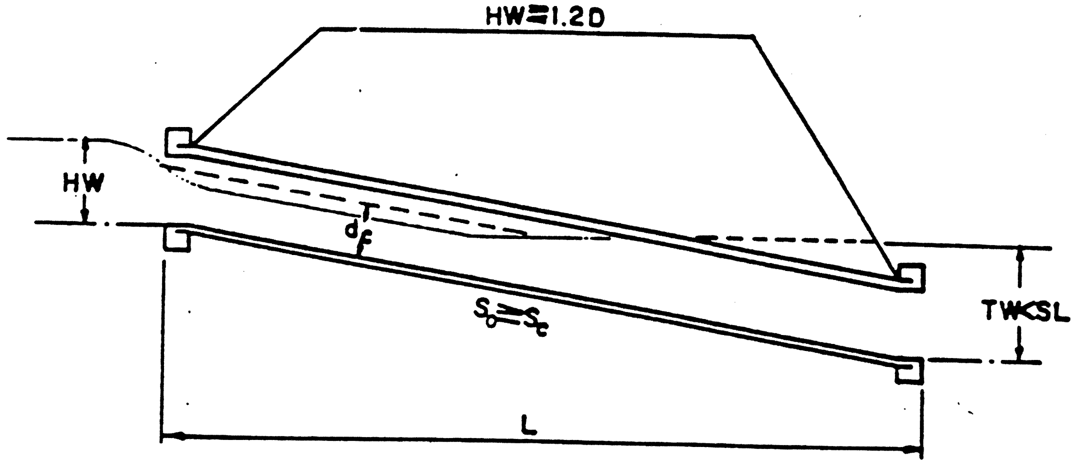
S_f = Friction slope or slope that will produce uniform flow. For Type II operation, the friction slope is based upon TW depth.

S_o = Slope of culvert in feet/foot.

L = Length of culvert in feet.

Type IIIA

Culvert Flowing Part Full With Inlet Control



Conditions

Entrance may be submerged or unsubmerged (HW≠1.2D) and the slope at design discharge is equal to or greater than critical slope ($S_o \geq S_c$). The tailwater depth is less than the vertical drop in the culvert from the upstream flow line to the downstream flow line ($TW<S_oL$) (tailwater elevation is lower than upstream flow line). Tailwater depth with respect to D is inconsequential as long as the above conditions are true.

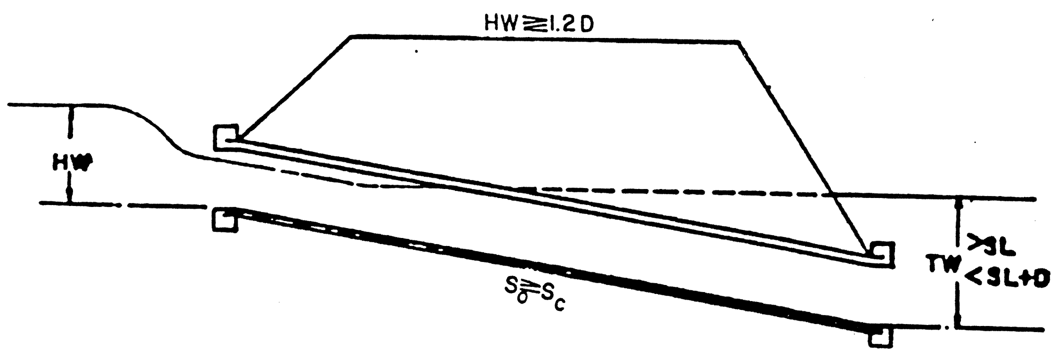
This condition is a common occurrence for culverts in rolling or mountainous country. The control is critical depth at the entrance for headwater values up to about 1.2D. Control is the entrance geometry for headwater values over about 1.2D.

Headwater is determined from empirical curves in the form of nomographs (see Figures 8-3, 8-8 and 8-9).

If the tailwater is greater than D, the outlet velocity is based on full flow at the culvert. If tailwater is less than D, the outlet velocity is based on uniform depth of the culvert.

Type IIIB

Culvert Flowing Part Full with Inlet or Outlet Control



Conditions

Entrance may be submerged or unsubmerged ($HW \neq 1.2D$), the slope at the design discharge is equal to or greater than critical slope ($S_o \geq S_c$), and tailwater depth is greater than the vertical drop in the culvert from the upstream flow line to the downstream flow line ($TW>S_oL$) (tailwater elevation is above the upstream flow line). Tailwater depth is less than the sum of the vertical drop in the culvert from the upstream flow line to the downstream flow line and D. Tailwater depth with respect to D is inconsequential as long as the above conditions are true.

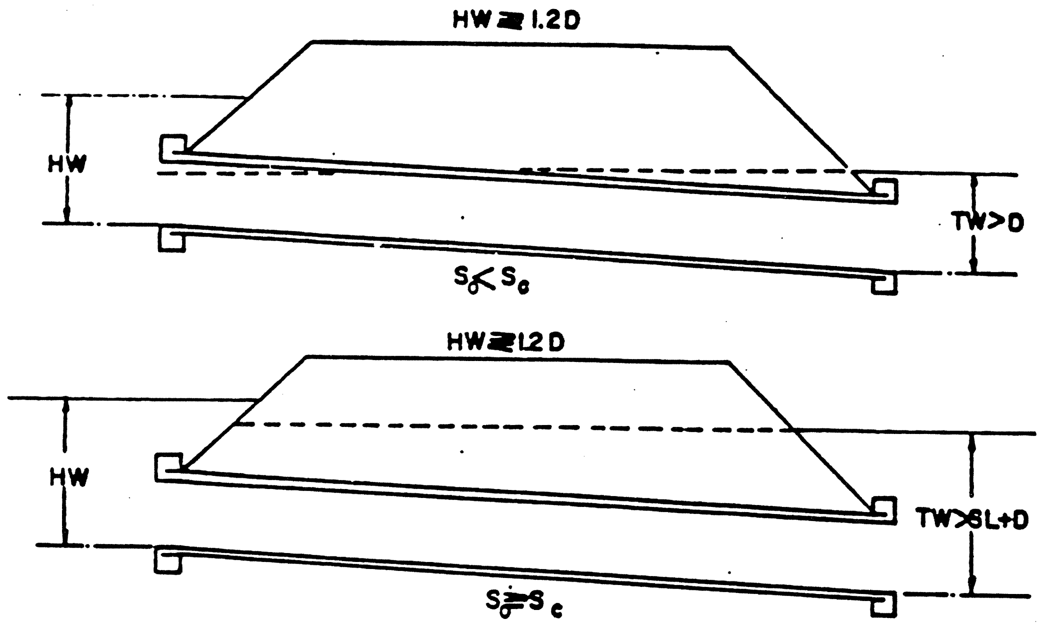
This condition is a common occurrence for culverts in rolling or mountainous country. The control for this may be at the entrance or the outlet, or control may vacillate between the two.

For this reason, headwater is determined for both entrance control and outlet control, and the higher of the two determinations should be used. Entrance control headwater is determined from empirical curves in the form of nomographs (see Figures 8-3, 8-8 and 8-9). Outlet control headwater is determined by procedures indicated for Type IVA or Type IVB (depending on tailwater depth with respect to D).

If tailwater depth is less than D, the outlet velocity should be based on tailwater depth. If tailwater depth is greater than D, outlet velocity should be based on full flow at the outlet.

Type IVA

Culvert Flowing Full With Submerged Outlet



Conditions

The entrance is usually submerged (HW>1.2D), the slope at the design discharge is less than critical slope ($S_o<S_c$), and tailwater depth is greater than D (TW>D). The tailwater completely submerges the outlet or the slope at design discharge is greater than or equal to critical slope ($S_o \geq S_c$), and tailwater is greater than the sum of the vertical drop in the culvert from the upstream flow line to the downstream flow line and D ($TW>S_oL+D$).

Most culverts flow with a free outlet, but depending on topography, a tailwater pool of a depth sufficient to submerge the outlet may form at some installations. Generally, these will be considered at the outlet. For an outlet to be submerged, the depth at the outlet must be equal to or greater than the diameter of pipe or height of box. The capacity of a culvert flowing full with a submerged outlet shall be governed by the following equation when the approach velocity is considered zero. Outlet velocity is based on full flow at the outlet.

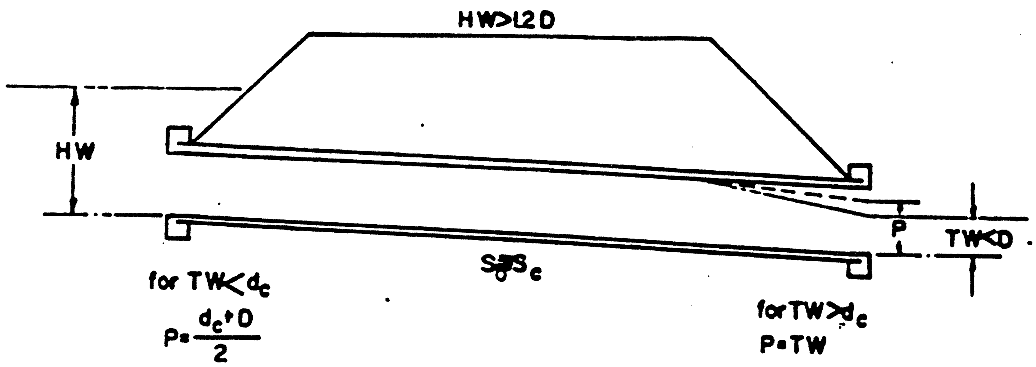
$$HW = H + TW - S_oL$$

where:

HW	=	Headwater depth above the invert of the upstream end of the culvert. Headwater depth must be greater than 1.2D for entrance to be submerged.
H	=	Head for culvert flowing full. $H = h_v + h_e + h_f$
h_v	=	Velocity head $\frac{V^2}{2g}$
V	=	Based on full flow in culvert.
h_e	=	Entrance head K_eh_v
h_f	=	Friction head = S_fL
S_f	=	is based on full flow in culvert.
TW	=	Tailwater depth in feet.
S_o	=	Slope of culvert in feet/foot.
L	=	Length of culvert in feet.

Type IVB

Culvert Flowing Full With Partially Submerged Outlet



Conditions

The entrance is submerged (HW>1.2D). The tailwater depth is less than D (TW<D).

The capacity of a culvert flowing full with a partially submerged outlet shall be governed by the following equation when the approach velocity is considered zero. Outlet velocity is based on critical depth if tailwater depth is less than critical depth. If tailwater depth is greater than critical depth, outlet velocity is based on tailwater depth.

$$HW = H + P - S_oL$$

where:

	HW	=	Headwater depth above the invert of the upstream end of the culvert. Headwater depth must be greater than 1.2D for entrance to be submerged.
	H	=	Head for culvert flowing full. $H = h_v + h_e + h_f$
	h_v	=	Velocity head $\frac{V^2}{2g}$
	V	=	Based on full flow in culvert.
	h_e	=	Entrance head K_eh_v

h_f = Friction head = $S_f L$

where S_f is based on full flow in culvert.

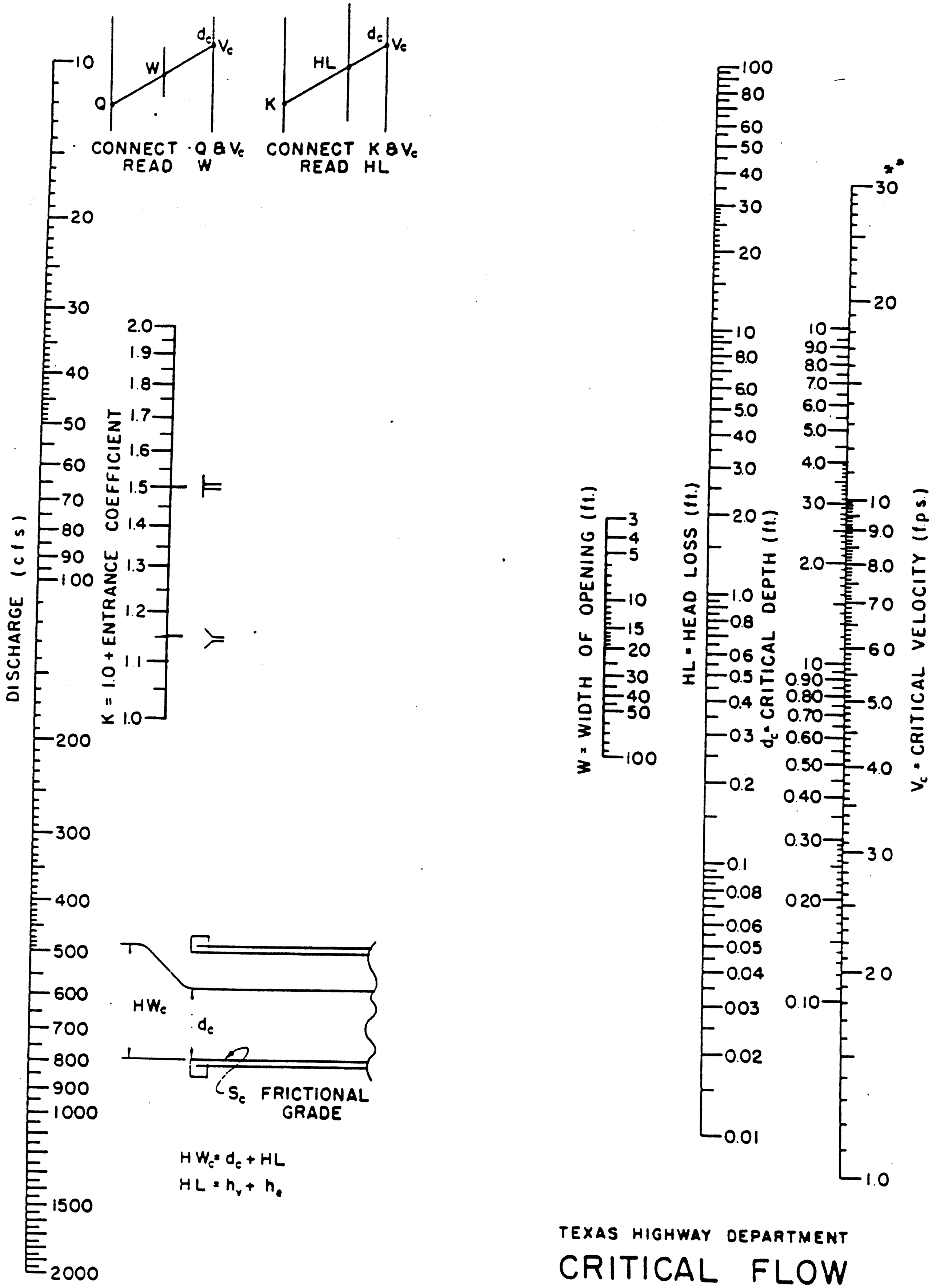
$P = \frac{(d_c + D)}{2}$ if TW depth is less than critical depth at design discharge. If TW is greater than critical depth, then P=TW.

d_c = Critical depth in feet.

D = Diameter or height of structure in feet.

S_o = Slope of culvert in feet/foot.

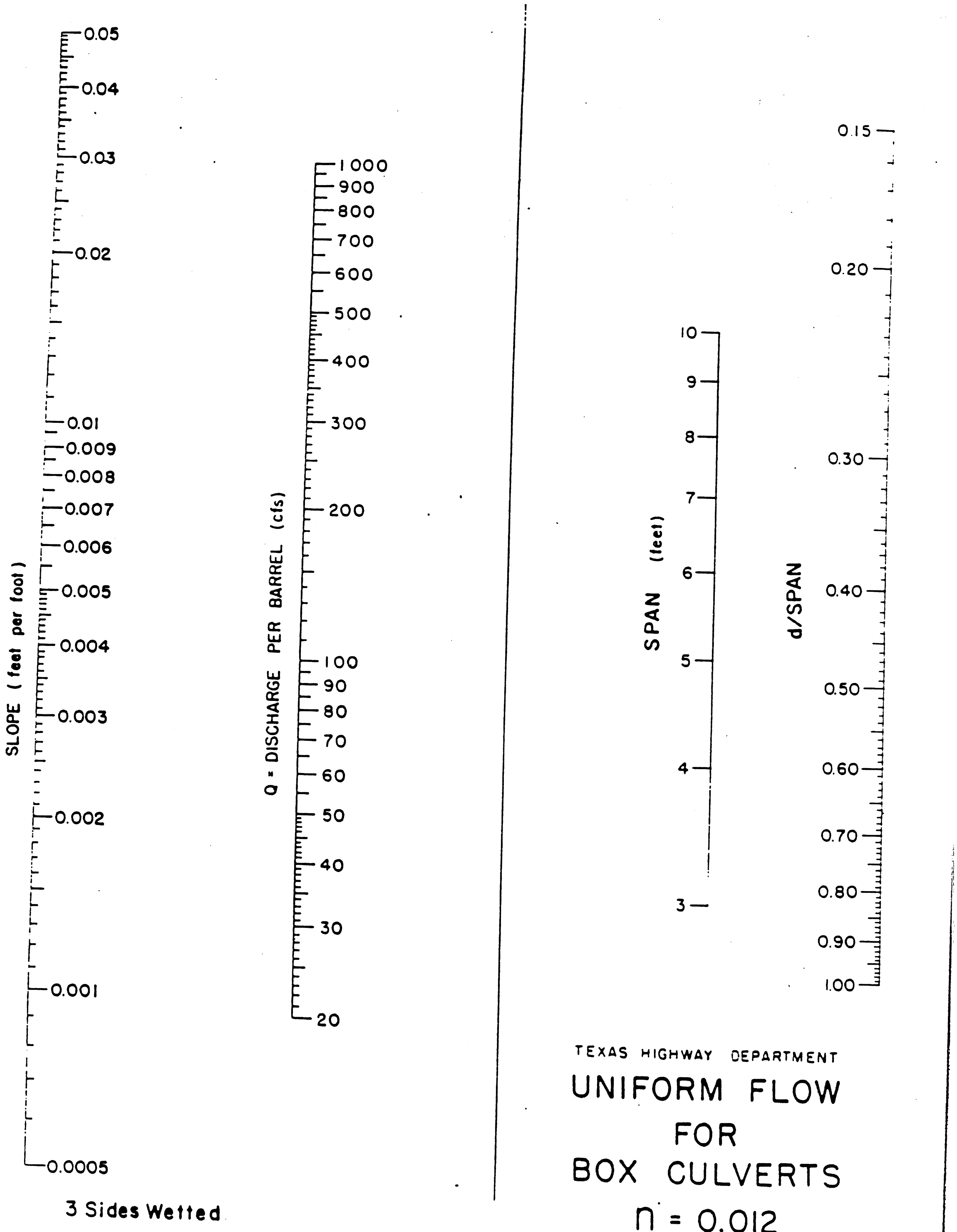
L = Length of culvert in feet.



TEXAS HIGHWAY DEPARTMENT

CRITICAL FLOW
FOR BOX CULVERTS
$n = 0.012$

Figure 8-1



SLOPE (feet per foot)
0.05
0.04
0.03
0.02
0.01
0.009
0.008
0.007
0.006
0.005
0.004
0.003
0.002
0.001
0.0005
Q = DISCHARGE PER BARREL (cfs)
1000
900
800
700
600
500
400
300
200
100
90
80
70
60
50
40
30
20
SPAN (feet)
10
9
8
7
6
5
4
3
d/SPAN
0.15
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00
3 Sides Wetted
TEXAS HIGHWAY DEPARTMENT
UNIFORM FLOW
FOR
BOX CULVERTS
n = 0.012

Figure 8-2

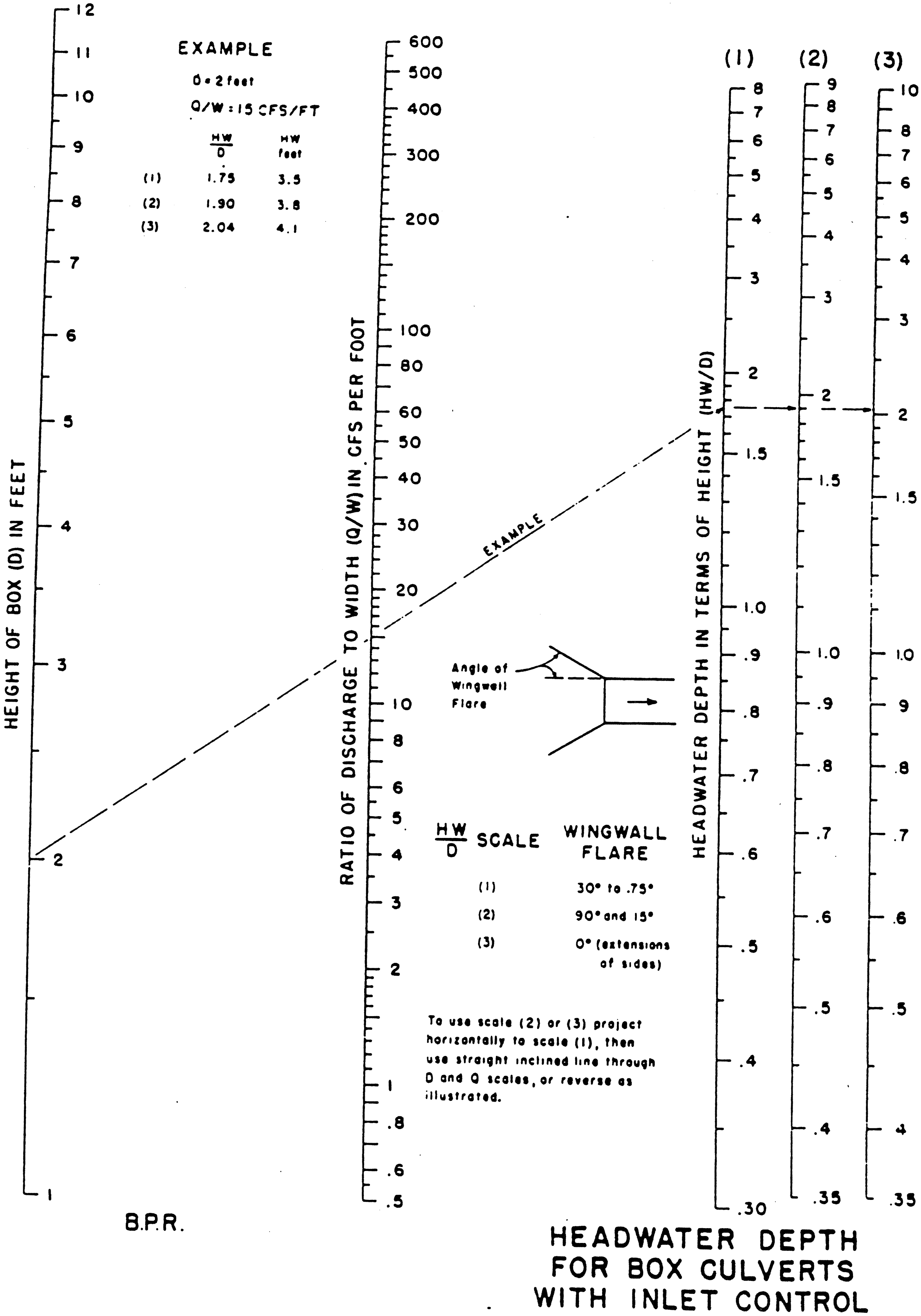
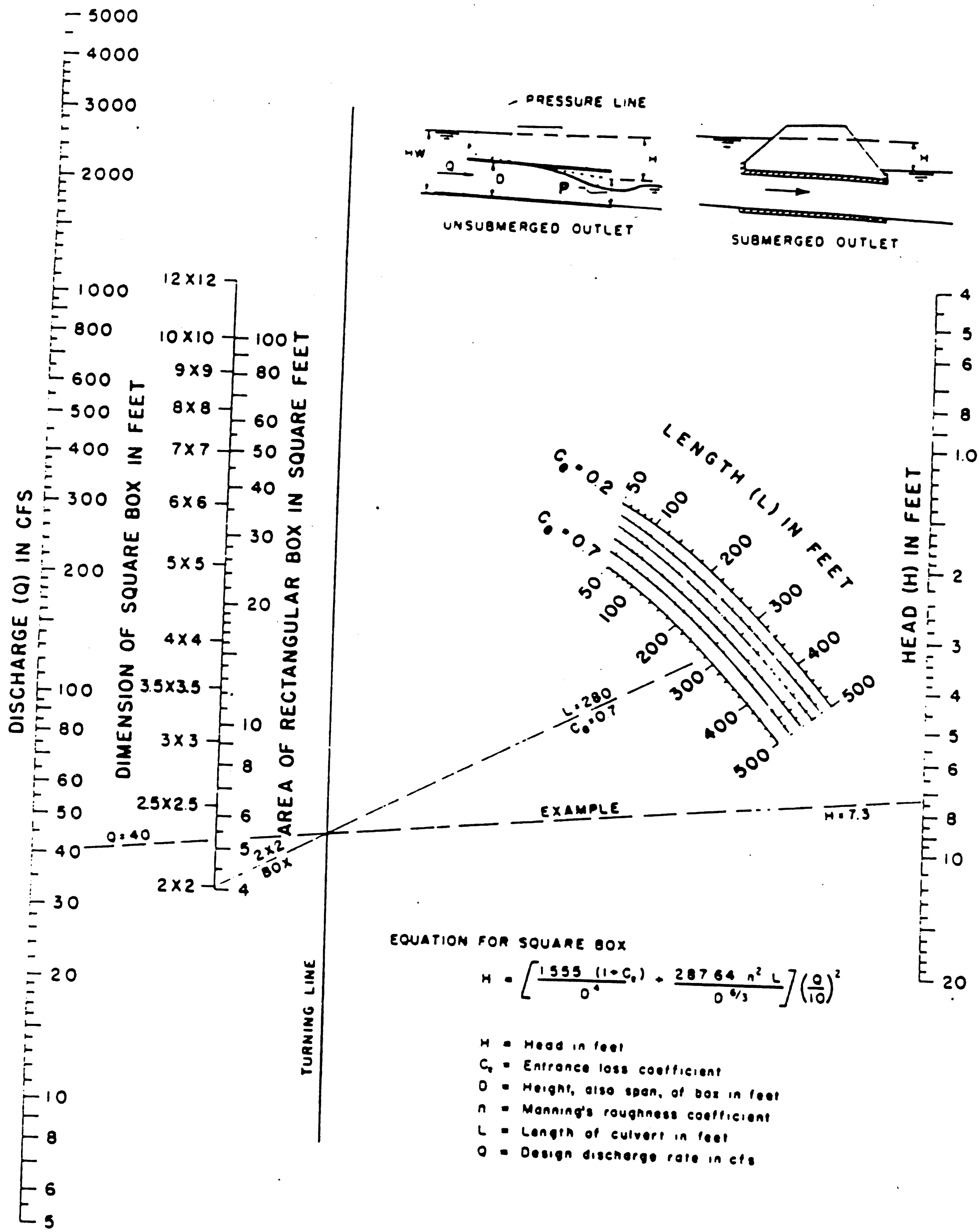

EXAMPLE
D = 2 feet
Q/W = 15 CFS/FT
HW/D HW feet
(1) 1.75 3.5
(2) 1.90 3.8
(3) 2.04 4.1
HEIGHT OF BOX (D) IN FEET
RATIO OF DISCHARGE TO WIDTH (Q/W) IN CFS PER FOOT
HEADWATER DEPTH IN TERMS OF HEIGHT (HW/D)
EXAMPLE
(1)
(2)
(3)
Angle of Wingwall Flare
HW/D SCALE WINGWALL FLARE
(1) 30° to 75°
(2) 90° and 15°
(3) 0° (extensions of sides)
To use scale (2) or (3) project horizontally to scale (1), then use straight inclined line through D and Q scales, or reverse as illustrated.
B.P.R.
HEADWATER DEPTH
FOR BOX CULVERTS
WITH INLET CONTROL

Figure 8-3



B.P.R.

HEAD FOR CONCRETE BOX CULVERTS FLOWING FULL
n = 0.012

Figure 8-4

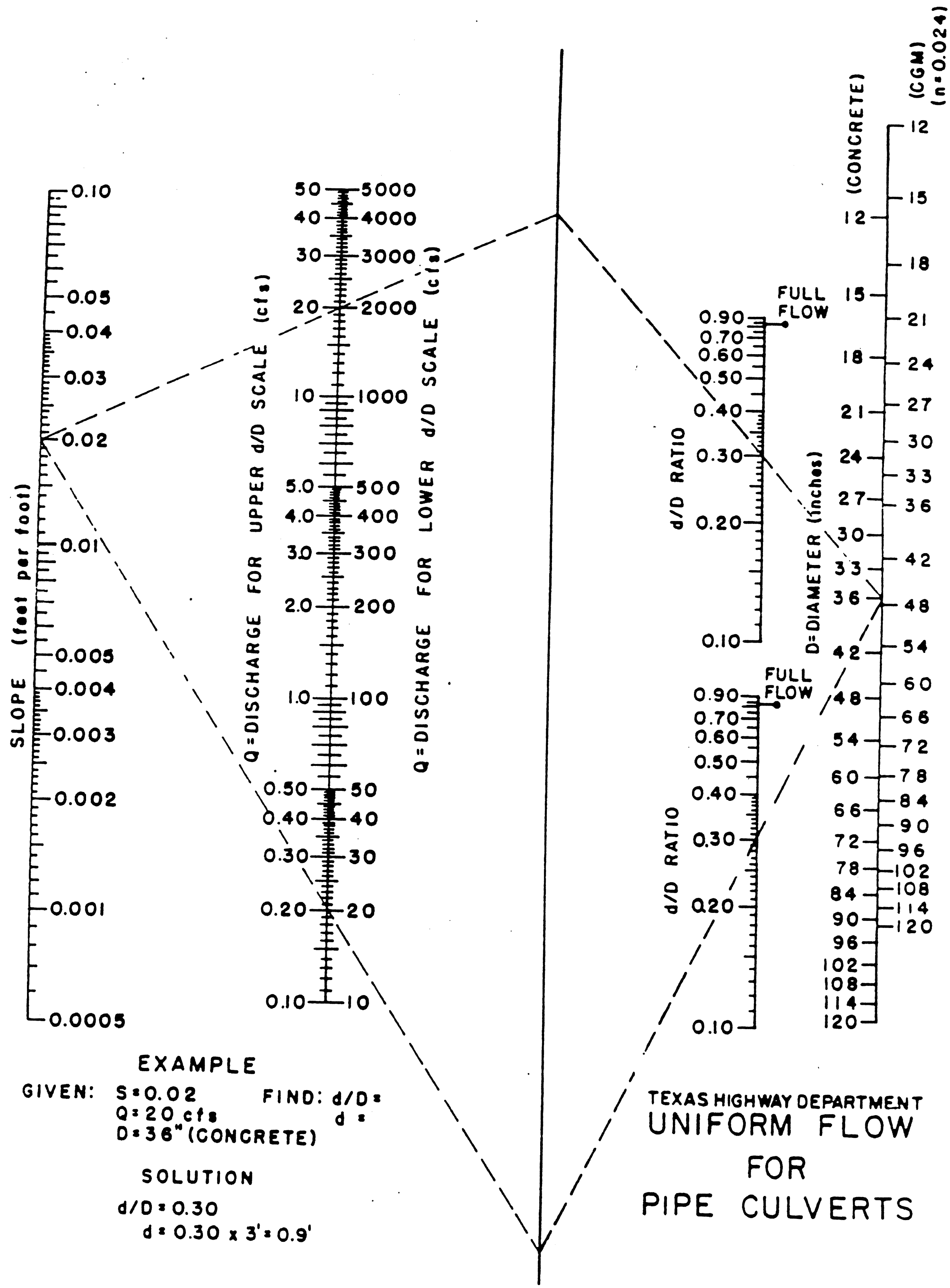
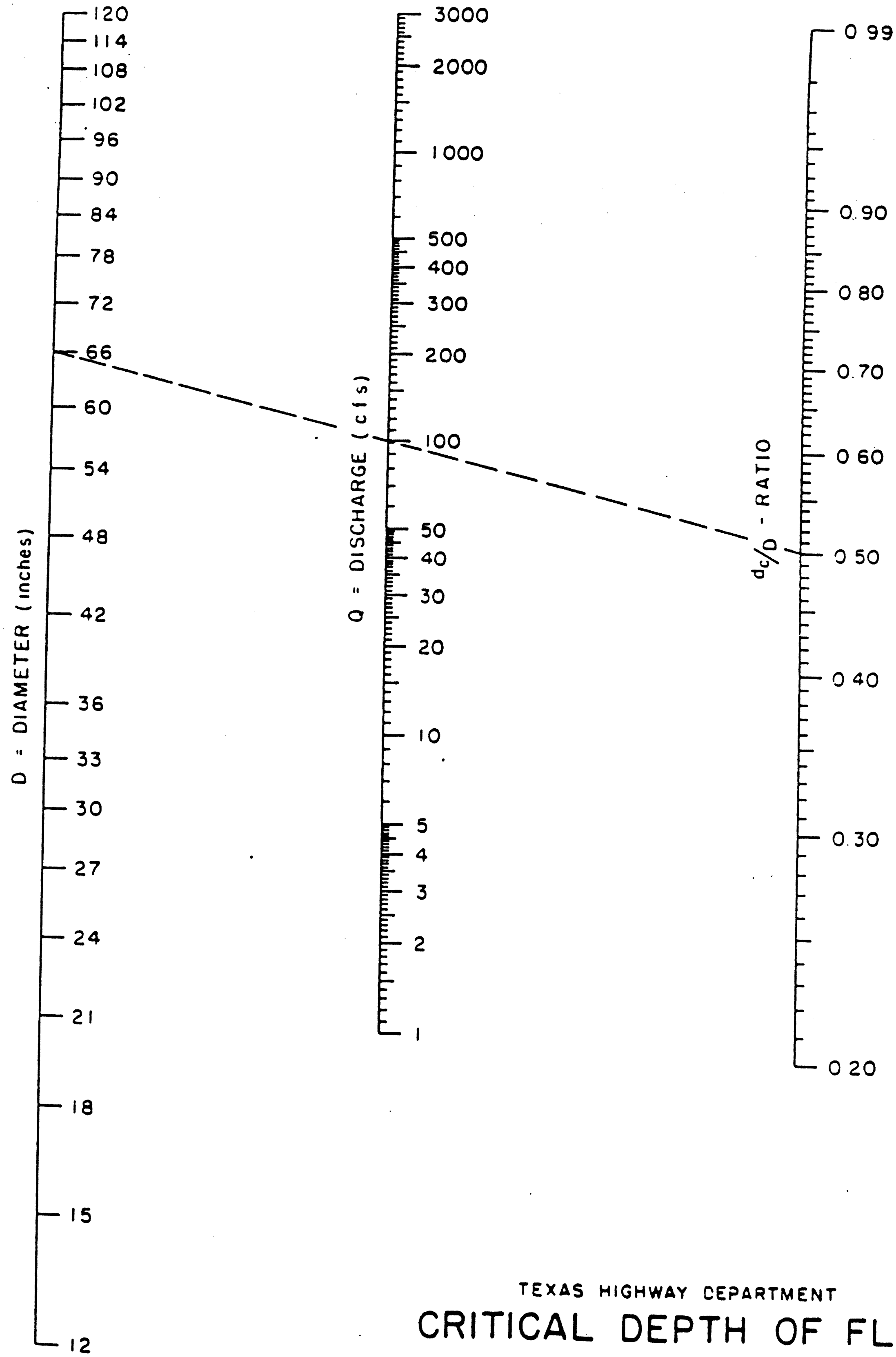

SLOPE (feet per foot)
0.10
0.05
0.04
0.03
0.02
0.01
0.005
0.004
0.003
0.002
0.001
0.0005
Q=DISCHARGE FOR UPPER d/D SCALE (cfs)
50 40 30 20 10 5.0 4.0 3.0 2.0 1.0 0.50 0.40 0.30 0.20 0.10
Q=DISCHARGE FOR LOWER d/D SCALE (cfs)
5000 4000 3000 2000 1000 500 400 300 200 100 50 40 30 20 10
d/D RATIO
0.90 0.70 0.60 0.50 0.40 0.30 0.20 0.10
FULL FLOW
d/D RATIO
0.90 0.70 0.60 0.50 0.40 0.30 0.20 0.10
FULL FLOW
D=DIAMETER (inches)
(CONCRETE)
12 15 18 21 24 27 30 33 36 42 48 54 60 66 72 78 84 90 96 102 108 114 120
(CGM) (n=0.024)
12 15 18 21 24 27 30 33 36 42 48 54 60 66 72 78 84 90 96 102 108 114 120
EXAMPLE
GIVEN: S=0.02
Q=20 cfs
D=36" (CONCRETE)
FIND: d/D=
d =
SOLUTION
d/D=0.30
d=0.30 x 3'=0.9'
TEXAS HIGHWAY DEPARTMENT
UNIFORM FLOW
FOR
PIPE CULVERTS

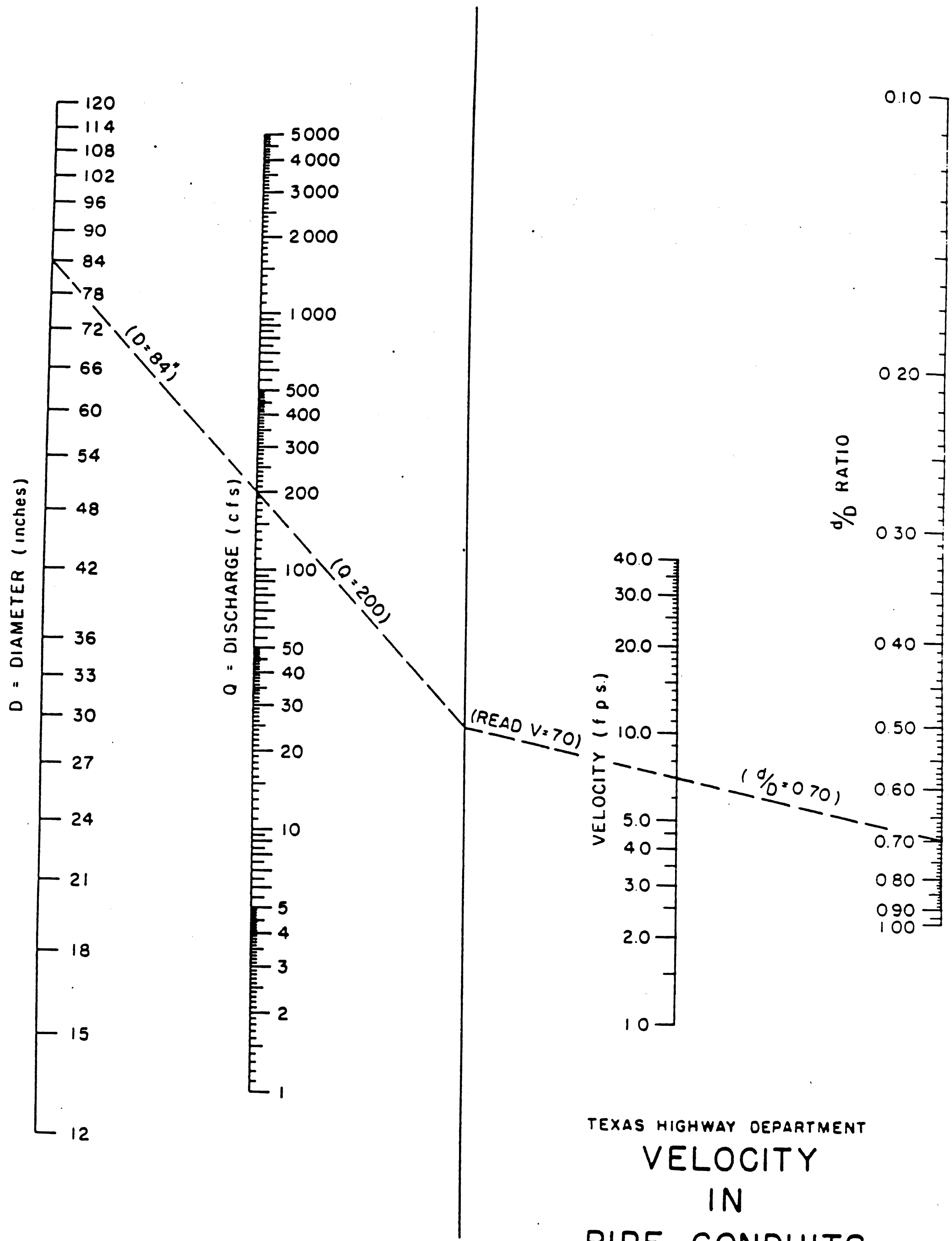
Figure 8-5



TEXAS HIGHWAY DEPARTMENT

CRITICAL DEPTH OF FLOW FOR CIRCULAR CONDUITS

Figure 8-6



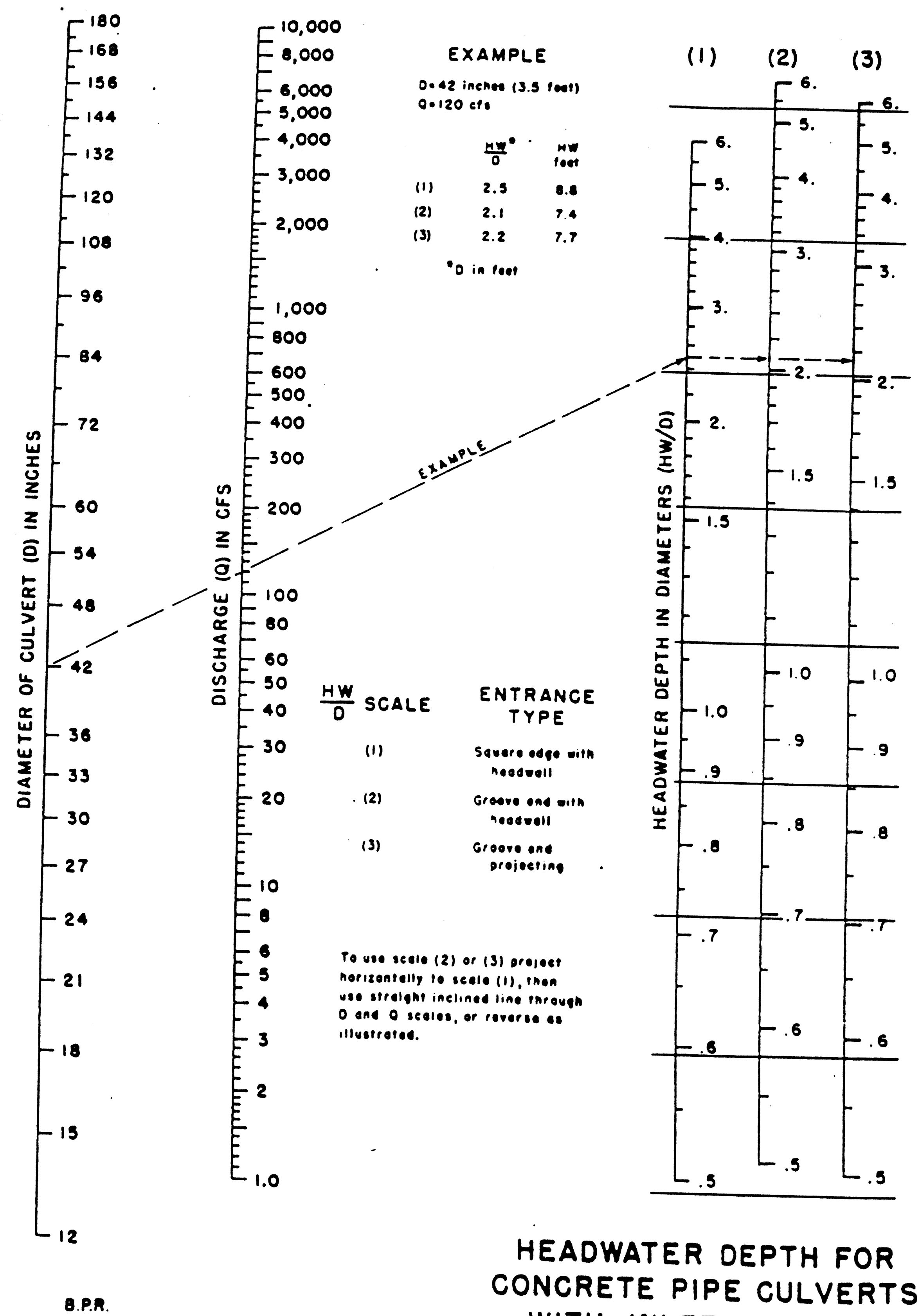
TEXAS HIGHWAY DEPARTMENT

VELOCITY IN PIPE CONDUITS

BASED ON

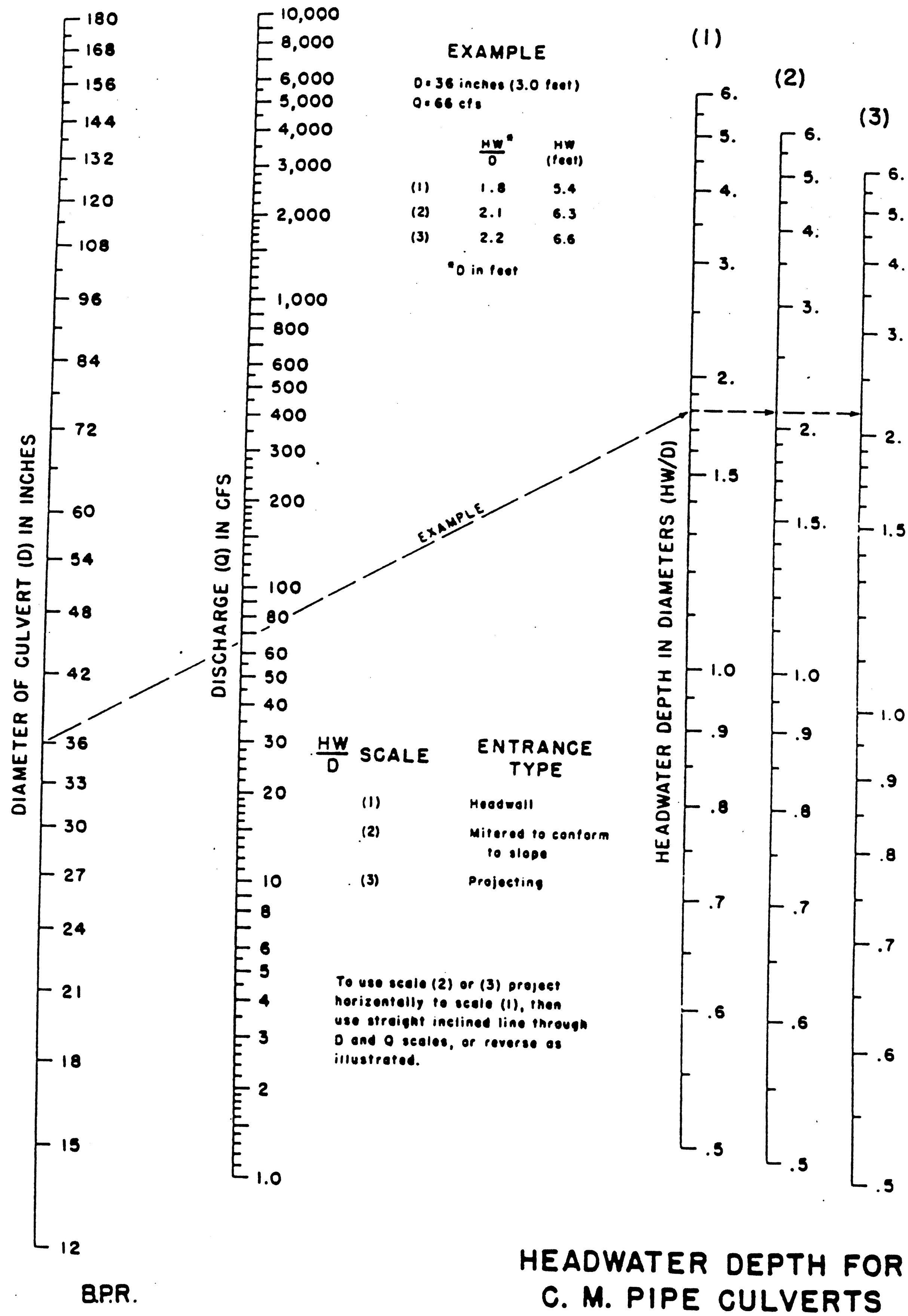
Q = VA

Figure 8-7



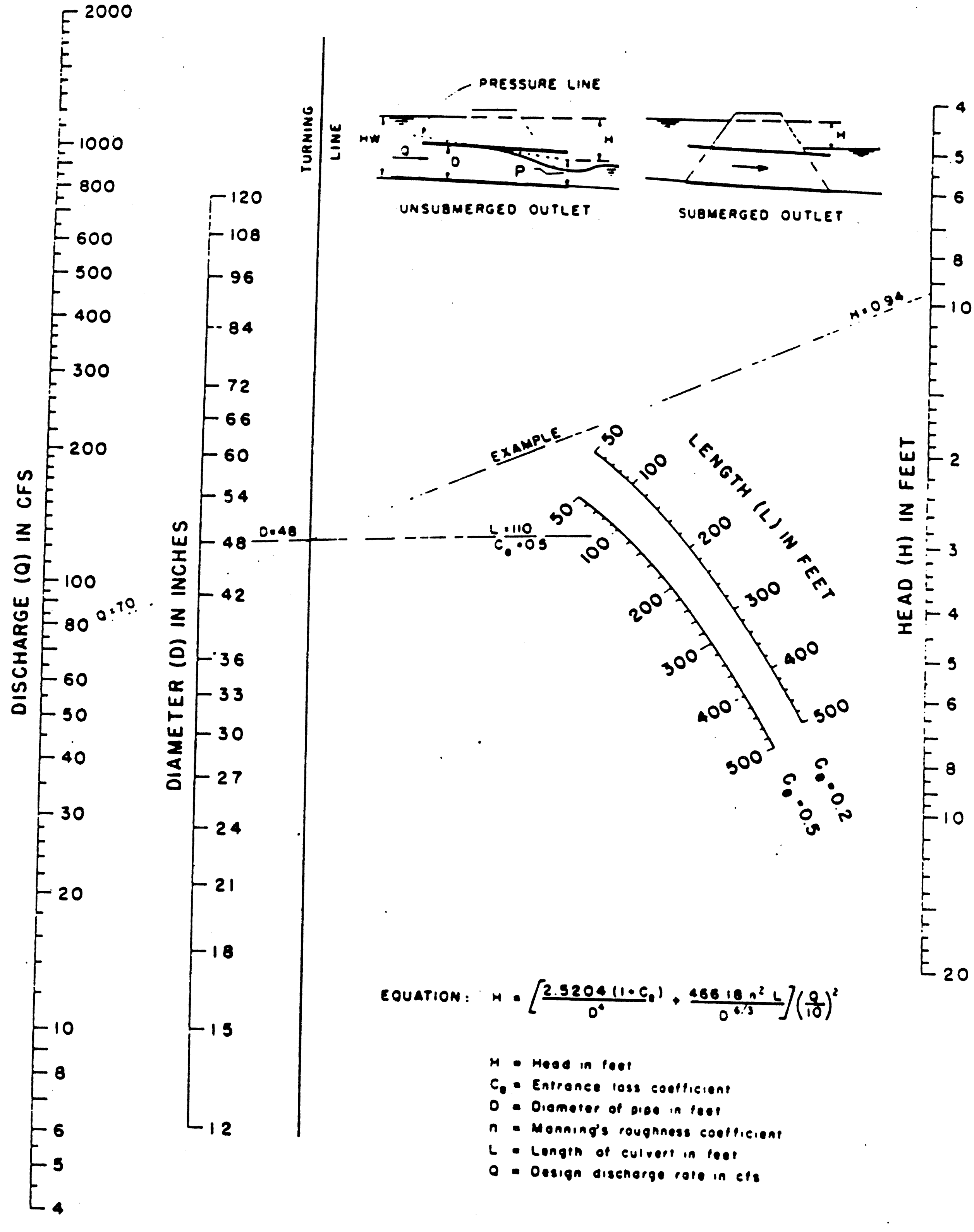
HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

Figure 8-8



HEADWATER DEPTH FOR C. M. PIPE CULVERTS WITH INLET CONTROL

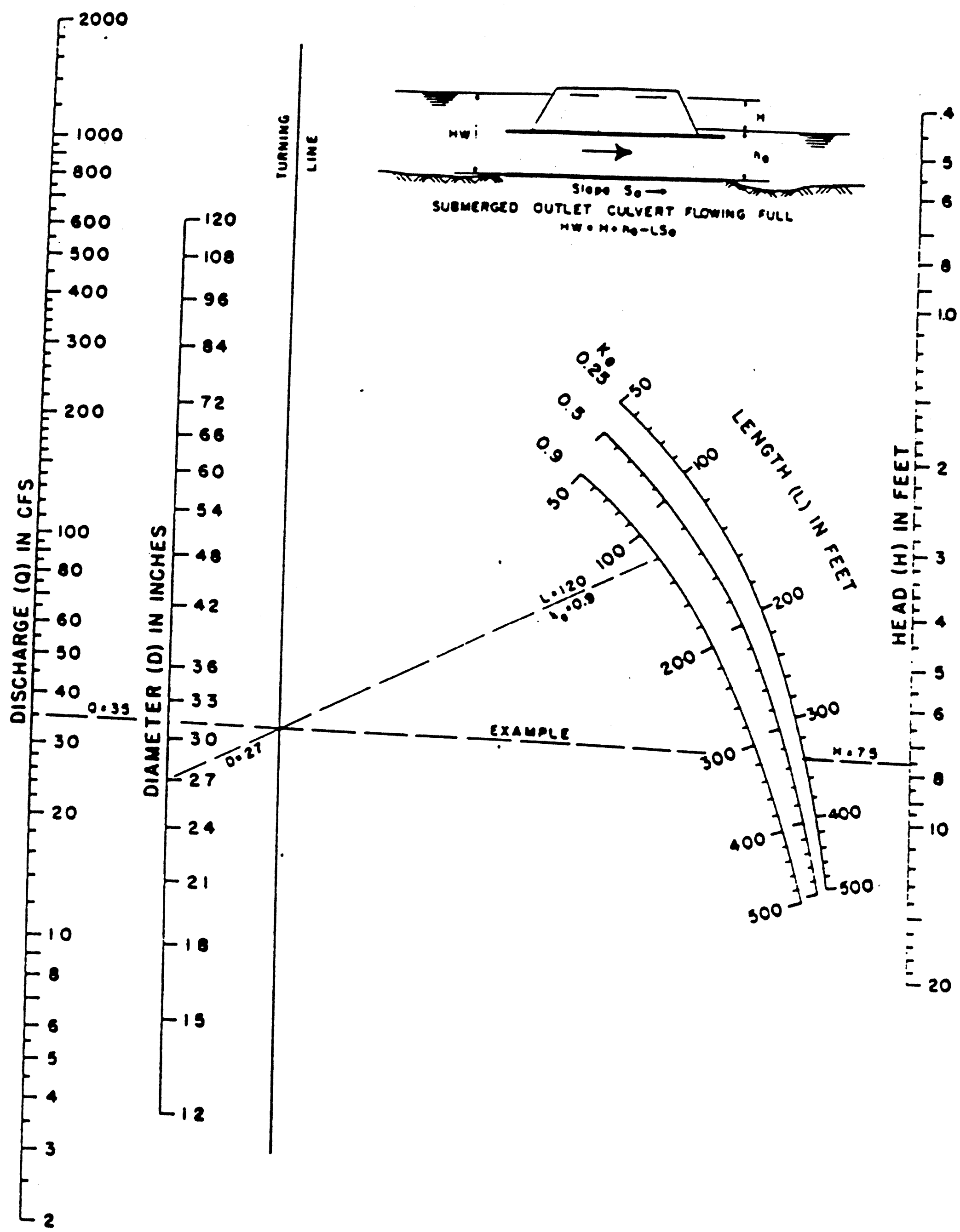
Figure 8-9



B.P.R.

HEAD FOR CONCRETE PIPE CULVERTS FLOWING FULL
n = 0.012

Figure 9-10



B.P.R.

HEAD FOR STANDARD C. M. PIPE CULVERTS FLOWING FULL n=0.024

Figure 8-11

9.0 CONSTRUCTION PLAN PREPARATION

9.1 GENERAL

The criteria and procedures outlined in this section provide a general guide to engineers who prepare construction plans for drainage projects in the Town of Addison. Engineers are encouraged to meet with the Town Engineer at the project concept level to discuss the specific design requirements of their project.

9.2 PRELIMINARY PLANS

It is recommended that a preliminary plan be completed in sufficient detail to allow review by the Town Engineer and to determine the problems which must be addressed during the final design. To complete this phase, topographic information is required to establish the general drainage patterns that impact the project. Topographic information may be obtained from on-the-ground field surveys, by aerial photogrammetric methods, by the use of the topographic maps available at the Town of Addison or from a combination of these sources.

Based upon the design criteria and procedures outlined in this manual, a preliminary layout of the existing and proposed facilities shall be prepared, including drainage areas, inlets, storm sewers and channels. The engineer shall review this plan with particular regard to the avoidance of utility conflicts where possible, and to establish the project requirements for easements or rights-of-way.

9.3 FINAL PLANS

During the final design phase, construction plans shall be drawn in ink on 24" x 36" mylar drafting film or equivalent, and shall be clearly legible. Complete drainage calculations shall be provided with the construction plans in accordance with the minimum requirements provided in this manual. Following approval of the construction plans by the Town Engineer, the design

- stationing and flow line elevations at all pipe size changes, grade changes, lateral connections, manholes and pipe connections;
- a profile of the storm sewer outfalls, including the water surface elevation at the outfall;
- lateral profiles where utilities are crossed; and
- all existing utilities, including the clearance with the proposed storm sewer.

10.0 STORM RUNOFF STORAGE

10.1 GENERAL

The introduction of impervious cover and improved runoff conveyance serves in many cases to increase flood peaks quite dramatically over those for existing conditions. When physical, topographic and economic conditions allow, channel improvements downstream of the development are often used to prevent increased flooding. When this is not feasible, a widely used practice is runoff detention or retention storage, wherein the storm volume is held back in the watershed and released at an acceptable rate. This section presents information on storage techniques, including guidance for the design of appropriate storm runoff storage facilities. The Town Engineer must be consulted concerning preferred watershed flood control strategies and alternatives.

10.2 STORAGE CLASSIFICATION

Storage systems may be classified as either on-line or off-line facilities. They may be designed for either detention or retention of stormwater.

- Retention Storage--In a retention storage facility, runoff is captured and released only after the storm event is over and the downstream water surface has subsided.
- Detention Storage--The vast majority of flood control storage is handled by detention facilities. The purpose of detention storage is to hold storm runoff back but release it continuously at an acceptable rate through a flow-limiting outlet structure, thus controlling downstream peak flows.
- On-Line Storage--An on-line storage facility is one in which the total storm runoff volume passes through the retention or detention facility's outflow structure.

- Off-Line Storage--An off-line storage facility is one in which storm runoff does not begin to flow into the storage facility until the discharge in the channel reaches some critical value above which unacceptable downstream flooding will occur. An off-line facility serves to store only the high flow rate portions of the flood event.

10.3 DESIGN PROCEDURES

The design process for a stormwater runoff detention facility generally proceeds as described below.

10.3.1 Hydrologic Design

1. Inflow Hydrograph--The design inflow hydrograph for the 100-year storm event must be determined by an appropriate hydrologic methodology. A discussion of the proper procedure for determination of a design inflow hydrograph can be found in Section 2.0 of this Manual.

2. Allowable Release Rate--The maximum allowable release rate from the detention facility must be determined. The outflow structure for a storage facility shall be sized such that the allowable release rate shall be limited to the 100-year design storm discharge based on existing conditions. The outflow structure capacity shall be determined using the methodologies discussed in Section 8.0 of this Manual.

10.3.2 Hydraulic Design

1. Preliminary Pond Sizing--A preliminary sizing of the detention facility can be carried out graphically as shown in Figure 10-1. A straight line is drawn from the origin to the maximum allowable outflow on the recession limb of the

hydrograph. The runoff volume depicted between the plotted hydrograph and the straight line approximates the necessary storage.

2. Preliminary Outflow Structure Sizing--The outflow structure may be sized preliminarily as follows:
   a. Determine the maximum allowable water surface elevation in the pond for the 100-year frequency inflow hydrograph.
   b. Estimate the flow line elevation for the outflow structure at the outlet.
   c. Estimate the size of the structure required to pass the allowable outflow rate based on a headwater elevation at the 100-year water level in the pond and a tailwater elevation at the estimated flow elevation in the outflow channel.
   d. Estimate the size of the overflow spillway required to pass the extreme storm events.

3. Design Tailwater Depth--In order to route the inflow hydrograph through the detention facility, a relationship must be established between the volume of storage in the pond and the corresponding amount of discharge through the outflow structure. In cases of outlet-controlled flow, this relationship is directly dependent on the elevation of the tailwater at the outlet of the outflow structure.

   For the purpose of establishing an outflow rating curve under outlet-control conditions, the design engineer must make a judgment as to what will be the proper tailwater level for use in calculating the storage-discharge relationship.

   In general, it should be noted that an unreasonably high choice for the tailwater depth will result in an oversized outflow structure, and thus minimize attenuation of the peak outflow, especially for smaller storm events. An unreasonably low choice for the tailwater depth will result in an undersized outlet structure and the risk that the pond will be overtopped during the design storm event. The Town Engineer shall be consulted as to preferred policy at the site of interest.

4. Final Sizing of Pond Storage and Outflow Structure--The inflow hydrograph should then be routed through the trial pond configuration (preferably using the Modified Puls routing option in HEC-1). A comparison can be made between the maximum discharge and the maximum allowable discharge. At this juncture, adjustments can be made in the pond storage and/or outflow structure in order to insure that the maximum discharge is not exceeded. Several routing iterations will probably be required for determination of the optimal storage volume and outflow structure.

   Detention or retention facilities shall be sized such that at least 1 foot of freeboard shall be maintained during the 100-year design storm.

5. Storm Sewer Hydraulic Gradients--The hydraulic gradients in storm sewers shall be determined using procedures outlined in Section 5.0 of this Manual. When storm sewers outlet into a detention facility, the starting water surface elevation for these calculations shall be the 100-year water level in the pond.

   In order to determine the 100-year flood levels in the detention facility, the 100-year inflow hydrograph is routed through the pond. It is recommended that the 100-year maximum water level in the main channel be used as the constant value of the tailwater elevation for determining the outflow structure rating curve for the 100-year design storm.

6. Allowances for Extreme Storm Events--Design consideration must be given to storm events in excess of the 100-year design storm. An emergency spillway, overflow structure or swale must be provided as necessary to effectively handle the extreme storm event. In places where a dam has been utilized to provide detention directly in the channel, due consideration must be given to the

consequences of a failure, and if a significant hazard exists, the dam must be adequately designed to prevent such hazards.

Detention facilities which measure greater than 6 feet in height are subject to 31 Texas Administrative Code (TAC) Chapter 299 (Subchapters A through E), which went into effect May 13, 1986, and all subsequent changes. The height of a detention facility or dam is defined as the distance from the lowest point on the crest of the dam (or embankment), excluding spillways, to the lowest elevation on the centerline or downstream toe of the dam (or embankment), including the natural stream channel. Subchapters A through E of Chapter 299 classify dam sizes and hazard potential, and specify required failure analyses and spillway design flood criteria.

7. Erosion Control--The erosional tendencies associated with an open channel are similar to those found in a detention pond. For this reason, the same types of erosion protection are necessary, including proper revegetation and pond surface lining where necessary. Proper protection must be provided at pipe outfalls into the facility, pond outlet structures and overflow spillways where excessive turbulence and velocities will cause erosion.

10.4 GENERAL REQUIREMENTS FOR DETENTION POND CONSTRUCTION

The structural design of detention facilities is very similar to the design of open channels. For this reason, all requirements from Section 7.0 pertaining to the design of lined or unlined channels shall also apply to lined or unlined detention facilities.

10.4.1 Pilot Channels

Construction of a pilot channel is required to facilitate complete drainage of the detention facility. The pilot channel shall meet the following specifications:

1. Unlined pilot channels shall be 2 feet deep at a minimum, have a minimum invert slope of 0.01 foot/foot and have maximum side slopes of 3:1.

2. Concrete pilot channels shall have a minimum depth of 2 inches and a minimum invert slope of 0.005 foot/foot.

3. The floor of the detention facility shall be graded toward the pilot channel at a minimum slope of 0.005 foot/foot and at a recommended slope of 0.0095 foot/foot.

10.4.2 Outlet Structure

Significant erosion protection at the outlet structure is required due to extreme headwater conditions and the erosive velocities which are typically present. The following erosion protection measures shall be required at the outlet structure:

1. Pipes, culverts and conduits shall be carefully constructed with sufficient compaction of the backfill. A geotechnical analysis shall be provided to support the proposed backfill requirements.

2. Reinforced concrete pipe used in the outlet structure shall conform to ASTM C96 Class III, with compression-type rubber gasket joints conforming to ASTM C443.

3. The use of pressure grouting around the outlet conduit should be considered where soil types or conditions may prevent satisfactory backfill compaction. Pressure grouting should also be used where headwater depths could cause backfill to wash out around the pipe.

10.5 GEOTECHNICAL INVESTIGATION

A geotechnical investigation shall be provided for at the proposed detention facility site to ensure proper design. This study shall be submitted to the Town Engineer and should address the following:

a. pond side slope stability;
b. suitability of excavated material for use as fill;
c. groundwater elevation;
d. potential for seepage through the dam and requirements for seepage control;
e. dam stability; and
f. effect on stability of adjacent structures and required control measures.

10.6 REQUIRED ANALYSES

The engineer shall be required to submit a summary of the technical calculations used in the design of any stormwater storage facility. This shall include at a minimum:

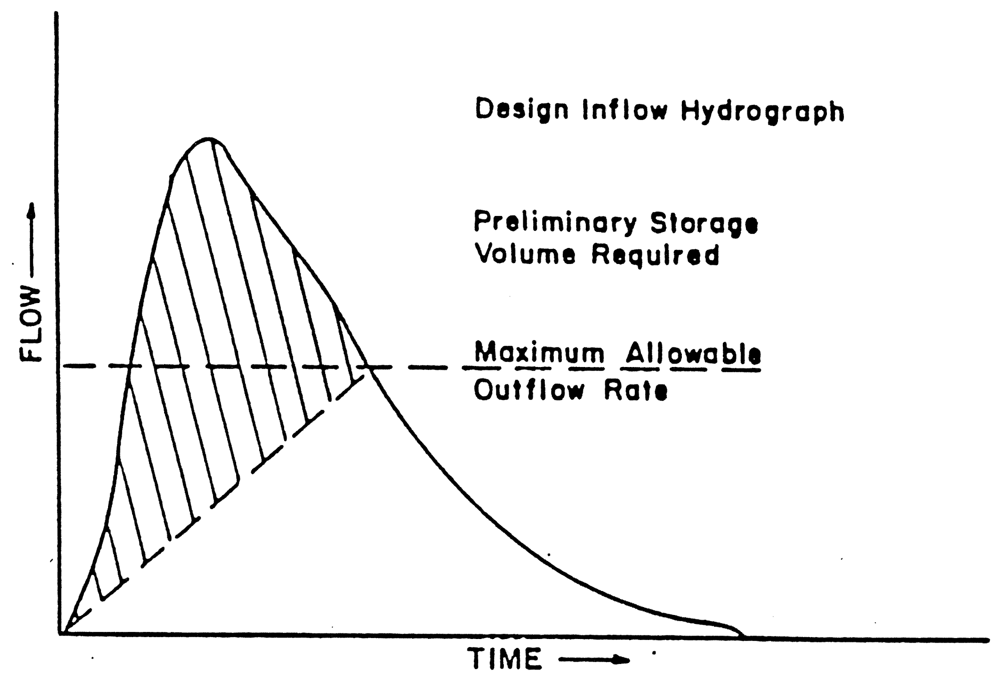
1. Hydrologic calculations for the 100-year existing and future condition flows.

2. A summary table delineating existing discharge, proposed discharge and post-detention discharge for each storage facility.

3. Routing calculations for the 100-year inflow hydrograph through the detention facility.

4. Calculations used to determine outflow structure rating curves.

5. Geotechnical Report as delineated in Section 10.4.



PRELIMINARY SIZING OF
STORAGE FACILITY

Figure 10-1

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APPENDIX A

GLOSSARY OF TERMS

APPENDIX A

GLOSSARY OF TERMS

Backwater	Water backed up in its course by an obstruction, an opposing current or by a larger body of water.
Crest	The surface on a dam or weir over which water is designed to spill.
Critical Flow	A state of flow characterized by minimum specific energy (depth + vel.^{2}/2g) and instability of depth. At critical flow, the velocity is equal to $(g[depth])^{1/2}$.
Critical Depth	The depth at which critical flow occurs.
Detention	The process of capturing stormwater runoff in a storage facility such as a pond or dammed-off section of channel while continuously releasing it from the facility at an acceptable rate through a flow-limiting outflow structure. The purpose of detention storage is to lessen peak flows downstream by controlling peak outflows at the detention site.
Flood Routing	The process for determining how the characteristics of a flood hydrograph change as it passes downstream through a channel or detention facility.
Floodplain	That portion of land area in and around a watercourse which becomes submerged, generally by water overflowing the stream banks, during a particular storm event. For example, the 100-year floodplain is defined as the land area which becomes submerged by streamflow during a 100-year frequency rainfall event.
Floodway	The natural floodway is the channel of a watercourse and those portions of the adjoining floodplain which are reasonably required to carry and discharge the floodwaters of a selected probability-of-occurrence flood. The designated floodway is the channel of a watercourse and that portion of the adjoining floodplain required to provide for the passage of a selected flood with an insignificant increase in flood stage above that of natural conditions. Normally, the 100-year flood (one that has a 1% chance of occurrence in any given year) should be considered as the selected flood. An

"insignificant increase" is established by State or local regulation.

Freeboard	The vertical distance between the design water surface level in an open channel and the top of the channel bank.
Head Loss	The "head" is a measure of the energy stored at a given point in a body of water. Therefore, head loss is a measure of the energy loss which may occur between two points in space. "Head loss" is generally used in the context of energy lost in a flow regime due to turbulence, friction, construction or expansion of the flow.
Hydrograph	A graphical depiction of the change in the rate of flow over time at a given point on a watercourse. The hydrograph is generally presented with time plotted on the x-axis versus flow rate plotted on the y-axis.
Hyetograph	A graphical depiction of rainfall intensity plotted versus time. Often, the rainfall intensity is assumed to be constant during each in a series of discretely defined time intervals.
Impervious Cover	Land cover, such as pavement or concrete, which does not allow the entrance or passage of water.
Open Channel	A watercourse where one water surface is exposed to atmospheric pressure.
Rainfall Intensity	The rate of rainfall. Rainfall intensity is generally measured in inches of rainfall per hour.
Retention	The process of capturing stormwater runoff in a storage facility. Retention differs from detention in that the stored runoff is not released until after the storm event has concluded and streamflows have subsided.
Special Flood Hazard Area	The land in a floodplain within a community subject to a 1% or greater chance of flooding in any given year.
Storm Duration	The length of time over which rainfall occurs in a given storm event.
Storm Frequency	The measurement of the magnitude of the rainfall in a given storm event presented in terms of the average interval of recurrence of such a storm. For example, a storm event of 100-year frequency is one of such magnitude that the average